NO. WW4
SEPTEMBER 1956

JOURNAL of the

Waterways and Harbors Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOLUME 82

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This Journal is published by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal.

AT,BD,CP,CO,HW,IR,SU,WW.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

September, 1956

Papers

Number

Alternatives to Stone in Breakwater Construction
by Reuben J. Johnson and Olin F. Weymouth 1059

The Harrison County Artificial Beach
by F. F. Escoffier 1060

Growth of Commerce: Tennessee and Cumberland Rivers
by G. M. Dorland and G. R. Bethurum, Jr. 1061

Lower Cumberland Project: Kentucky and Tennessee
by Albert E. Dykes 1062

Arrangement of Groins on a Sandy Beach
by Shositiro Nagai 1063

Discussion 1068

DIVISION ACTIVITES

Newsletter 1956-21

NOTE: Contents also includes
Design Considerations for a New Lock at Wilson Dam
by Robert A. Monroe and George P. Palo 1069

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

ALTERNATIVES TO STONE IN BREAKWATER CONSTRUCTION

Reuben J. Johnson,[1] A.M. ASCE, and Olin F. Weymouth[2]
(Proc. Paper 1059)

ABSTRACT

The type of breakwater construction employed depends on economic considerations, depth, and salinity of water, severity of storms and availability of materials. The difficulty in obtaining armor stone has resulted in investigation of alternatives such as concrete castings in the form of rectangular blocks, tetrahedrons, and tetrapods. Advantages of this type of construction indicate increasing feasibility in future breakwater work.

INTRODUCTION

A breakwater is defined as a structure employed to break or dissipate the force of waves and thus prevent their incidence on an area it is desired to protect. A jetty, on the other hand, is a structure extending into a body of water to intercept littoral drift or to direct and confine the stream or tidal flow to a selected channel. The basic principles involved in the design of either a breakwater or a jetty are essentially the same; consequently, the subsequent discussion is generally applicable to either of the foregoing types of structures.

The design of structures to withstand the continual pounding of the sea has taxed the ingenuity of mankind for thousands of years. The earliest breakwaters were of the rubble-mound type constructed of relatively small stone limited in size to that which could be handled with the construction equipment of the time. The stone was placed in a random manner to a crest height sufficiently above water to provide the desired protection. It soon became evident that the sea slopes were too steep or the stones of insufficient size to resist forces delivered by storm waves. The action of high waves, which lowered the top of the mound and flattened the seaward slope, made it necessary to replenish the mound until an equilibrium slope was reached. This

Note: Discussion open until February 1, 1957. Paper 1059 is part of the copyrighted Journal of the Waterways Division of the American Society of Civil Engineers, Vol. 82, No. WW 4, September, 1956.

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slope was found to vary from 1 on 5 to 1 on 10 on the seaward side within range of the worst attack.

Types of Breakwaters

Within the last century, breakwater construction has expanded to include, in addition to the rubble-mound type, the vertical face and composite types. Economic considerations, depth and salinity of water, severity of storms, and availability of materials, are general features entering into the selection of the type to be used.

Major examples of the vertical-face type are the masonry-wall breakwater; the sheet-pile breakwater constructed of timber, concrete or steel; caisson breakwater constructed of steel or concrete, and timber or concrete crib breakwaters. In general, the vertical-face type provides the greatest variety of alternatives for stone in breakwater construction. It is used principally where the wave pressure is that of a standing wave in contrast to the breaking wave.

A second general type, the composite breakwater, consists of a masonry wall placed on an underwater rubble mound. The rubble mound is used either as a foundation for the wall or as a main substructure surmounted by a wall superstructure with a vertical or inclined face. Its use often provides economic advantages where the foundation is soft and subject to scour, or at locations having large tidal variations where excessive quantities of stone would be required to construct a rubble-mound structure.

The third general type of breakwater, the rubble mound, consists of a structure having an interior core of sand, clay, or small rock protected with cap rock. Its chief advantages are that damage is easily repaired, settlement of the structure results in increased stability rather than failure, and the reflected wave action is less severe than from a solid wall structure. The chief disadvantage is the large quantity of material required. In general, natural rock obtainable in the vicinity of the breakwater has been used in construction. However, because of the difficulty in obtaining large cap or armor stone, weighing from 10 to 20 tons, and the high cost thereof, it has become feasible to substitute alternatives such as concrete castings in the form of either rectangular blocks, tetrahedrons, or tetrapods. The jetties at the entrance of Humboldt Harbor, California, provide an example of the use of tetrahedrons and concrete blocks. Figures 1 and 2 are diagrams of the cross-section of the north jetty. Figures 3 and 4 show condition of north jetty after 15 years in operation.

Design of Rubble-Mound Breakwaters

Since the primary purpose of this paper is to review recent methods and materials that have been developed as alternatives to the rubble-mound breakwater a brief review of the principles of design are in order. In the years previous to the 1930's the design of rubble-mound structures was, in general, based upon the behavior of existing structures and the experience of the designing engineer. No attempt was made to determine theoretically the size of stone or the slope to be used until 1933 when Professor Don Eduardo de Castro developed a formula based upon the premises that (a) the destructive action of a wave is proportional to its energy; (b) the weight of the stone

necessary to withstand the wave energy varies directly as density of the stone in air and inversely as the cube of its density submerged in water; and (c) the stability of stone subject to wave action is inversely proportional to some geometric function of the slope upon which it rests. Professor Ramon Iribarren Cavanilles, in 1938, published a formula incorporating the foregoing elements, which has been used for most breakwater design in recent years. One of the limitations of this formula however, was the empirically determined coefficient K which included all of the unevaluated variables. This formula has been modified recently by Mr. R. Y. Hudson of the Corps of Engineers Waterways Experiment Station at Vicksburg, using the same assumptions and force diagram as Iribarren to obtain the following expression:

$$W = \frac{K' \, w \, S_f^{\,3} \, S_r \, \mu^3 \, H^3}{(\mu \cos\alpha - \sin\alpha)^3 \, (S_r - S_f)^3}$$

in which

W = weight of individual stone in tons of 2,000 pounds.
K' = A variable dimensionless empirically determined coefficient.
w = unit weight of fresh water.
S_f = specific gravity of the fluid in which the breakwater is located.
S_r = specific gravity of the rock.
μ = effective coefficient of friction rock on rock.
H = total height of wave which breaks on the breakwater, in feet.
α = angle the slope makes with the horizontal.

This formula is currently in use by the Corps of Engineers in breakwater design. Curves for values of K' as related to the side slope and the ratio of the depth of water to the wave length (d/L); and for wave height, slope and W/K' have been developed from small-scale experiments. These curve relationships are being further verified and refined as new information is developed by existing breakwaters and model analysis.

West Coast Breakwater Construction

The coast line and sea characteristics along the west coast of the United States have been major factors favoring construction of rubble-mound type breakwaters. All major harbors along this coast, with the exception of San Francisco Bay, require breakwaters or jetties. To illustrate the progressive development of breakwater construction typical of this area, including a growing need for alternatives for breakwater stone, the structure at Crescent City, California, is discussed herein in detail. Figures 5 and 6 show oblique and side views respectively of this breakwater. Figure 7 is a plan of the breakwater.

Crescent City is located about 17 miles south of the Oregon State Line. The harbor is used primarily by lumber, petroleum and fishing interests employing light-draft vessels. The breakwater has been progressively lengthened and improved over the past 35 years. The initial length of 2,245 feet was constructed during the period 1920-25 and extended to 3,000 feet in 1930. Side slopes ranged from 1 on 1 to 1 on 1-1/2 on the harbor side and from 1 on 1-1/2 to 1 on 2 on the ocean side. A 10-foot thick blanket of 10-ton

armor stone extends from about elevation +1 on the harbor side to a maximum of -16 m.l.l.w. on the ocean side. A concrete cap was added to 1929-30 which brought the crest elevation to a +14 m.l.l.w. This 3,000-foot structure appears to have been adequately constructed because maintenance was required only once during the succeeding 17 years. It is to be noted that, within this section, depths do not exceed 25 feet and the structure is somewhat protected by offshore rock islands.

The breakwater was extended seaward another 1,000 feet in 1947-48. Slopes on the harbor side were maintained at 1 on 1-1/4 and on the ocean side at 1 on 1-1/2. The average stone weight was increased to 12 ton. Storms during the winter of 1948-49 caused a loss of armor stone on the harbor slopes and a partial breach in this extension. In view of these damages the cross-section of the further extension of the breakwater was modified by flattening the upper portion of the slopes to 1 on 1-1/2 on the harbor side and 1 on 1-3/4 on the ocean side. A contract for the final 1,800-foot section of breakwater, to bring the total length of the structure to 5,800 feet was initiated in 1949. By the end of the first working season approximately 200 feet had been constructed. In the following winter season the structure seaward of station 30+00 again suffered moderate to severe damage consisting of loss of armor stone on the harbor side and a near breach at two locations.

Subsequently, to reduce the amount of further damage and the number of waves overtopping the structure, a concrete cap, 22 feet in width with crest elevation at +20 MLLW was added for almost the full length of breakwater. Figure 8 is a view directed along this cap looking toward the seaward end. Thickness of the cap varied from 3 to 5 feet with the concrete specified to penetrate not less than 3 feet into the rubble. Steel dowels were employed to provide additional bond between the cap and rubble structure. Transverse construction joints at approximately 40-foot intervals formed the cap structure into individual blocks weighing roughly between 200 and 300 tons.

Storms during the 1950-51 winter season, possibly the most severe in at least 20 years, displaced this concrete cap and all stone to about MLLW in the outer 450 feet of the structure and caused the loss of considerable quantities or armor stone from the harbor slopes seaward of station 30+00. It is noted that forces exerted by the overtopping storm waves were the most critical in all of these storms. These forces caused the harbor side stones to move outward and thus undermine the cap.

The damage sustained during 3 successive years demonstrated the impracticability of any further extension of the breakwater on the existing alignment. Furthermore, reconstruction of the outer 450 feet seaward of station 37+00 was considered infeasible.

A detailed study was made in 1952 to develop a breakwater section based on the latest design criteria. The study included (1) a refraction analysis of waves of various periods which approach the harbor from the critical sector between NNW and SW, (2) an analysis of diffraction patterns to determine the most suitable alignment of breakwaters, and (3) a determination of breakwater slopes based on interpretation of the modified Iribarren formula.

It was determined that redirecting the breakwater from station 37+00 in an easterly direction for a distance of about 1,000 feet would provide improved protection and still permit maximum harbor development under foreseeable future conditions. The extension, in addition, would not be subject to as severe wave action as the breakwater extension on the original alignment. Analysis of storm-wave heights and frequencies, as well as maintenance

costs, indicated that the strengthening of the existing breakwater from station 30+00 to 37+00 and the realigned extension should be designed, whenever practicable, to withstand the maximum height of the short-period waves (periods less than 10 seconds) that could exist at the breakwater. The height of such waves was determined to be controlled solely by the depth and bottom slope seaward of the breakwater and was evaluated at 33 feet for the section to be strengthened and 23 feet for the major portion of the realigned extension.

The average maximum size armor stone that can be economically quarried in the Crescent City area is about 12 tons. Substituting this size stone in the modified Iribarren formula required breakwater slopes much flatter than are feasible, either physically or economically, to meet the desired design-wave height criteria of all sections seaward of station 30+00, except at the innermost portion of the extension. Weather and sea conditions at Crescent City have not permitted the use of floating plant, and thus all stone must be placed by equipment operating from the cap of the breakwater. The maximum distance that stone may be placed from the centerline of the breakwater is about 120 feet. Under these physical limitations it was determined that the existing breakwater seaward of station 30+00 should be strengthened by placing additional stone on the ocean side to provide slopes of 1 on 4 above MLLW. Crest elevation remained as previously constructed at +20.

The strengthening, as outlined above was accomplished in 1952-54. Work on the directed extension was initiated immediately thereafter and to date approximately 450 feet have been constructed. The ocean slopes on the extension vary from 1 on 2-1/2 to 1 on 3-1/2 above elevation -10 MLLW and 1 on 1-1/2 below this elevation; harbor slopes are 1 on 1-1/2 above elevation -6 and 1 on 1-1/4 below said elevation. The concrete cap has been omitted from the redirected section.

Recognizing the limitations of the rubble-mound structure as constructed, and faced by an increasing shortage in available stone supplies, the San Francisco District of the Corps of Engineers has continued its efforts to employ substitutes for rubble-mound construction. One of the most promising possibilities appears to be the use of concrete tetrapods as a replacement for armor stone. The tetrapod, illustrated in Figure 7 was developed by the NEYRPIC Hydraulic Laboratories in Grenoble, France, and may be described as an integrally cast, four-armed, concrete block, any two arms of which form an angle equal to that formed by any other two arms. The arms are shaped as truncated cones and are joined to a central core. The tetrapod has reportedly been employed with success abroad, but, as far as known, has not been used in the United States for either breakwater or jetty construction.

Model tests of idealized breakwater sections constructed of tetrapods were recently completed at the Corps of Engineers Waterways Experiment Station. Tests involved determination of design waves for no damage, and values of K' for use in Iribarren's formula over a range of slopes and d/L ratios with breakwaters constructed of two, three, and four layers of tetrapods. The test sections were based on possible application to Crescent City where it was considered the breakwater would be constructed with a stone core enveloped by several layers of tetrapods. Comparison of the calculated K' values indicated that stability of the breakwater would not be substantially improved by the use of more than two layers of tetrapods. As in the case of stone, the tests indicated that K' values varied appreciably with slope and to a much lesser degree with the ratio d/L. Average values of K' ranged from

0.003 for a 1 on 1-1/4 slope to 0.036 for a 1 on 3 slope.

A comparison of weights of individual tetrapods and armor stone, which have been calculated using the Iribarren's modified formula and the foregoing experimentally determined average K' values, is shown in Figure 9. The computations are further based on coefficients of friction of 1.10 for tetrapods and 1.05 for stone and specific gravities of 2.40 for tetrapods and 2.56 for stone. Expressing efficiency as the ratio of design-wave heights, it may be seen that under the foregoing conditions, tetrapods could be from 40 to 70 percent more efficient than armor stone for comparable side slopes and weight.

On the basis of test results, plans are presently under way to complete the Crescent City breakwater with tetrapods. The breakwater section would consist of a core of Class "C" stone (5 pounds to 2 tons), enveloped by a double layer of 4.5-ton tetrapods and capped with a double layer of 15-ton tetrapods; harbor slopes would be 1 on 1-1/2 and 1 on 1-1/4 above and below elevation -6 MLLW, respectively; and seaward slopes would be 1 on 2 and 1 on 1-1/2 above and below elevation -10 MLLW, respectively. The crest section (20 feet in width at elevation +20) would be constructed of stone in order to provide a roadway for placement of materials from the crest.

Tetrapods are also being considered for the repair of several breakwaters in the Hawaiian Islands. At Kahului Harbor on the island of Maui, the design wave has been estimated at 34 feet—a height for which structural stability could hardly be obtained with rubble-mound construction. With 60-ton stone, slopes of 1 on 10 are indicated by the formula, whereas the design criteria could be met with 29-ton tetrapods and 1 on 3 slopes. Nawiliwili Harbor breakwater repair on the island of Kauai also is in the realm of feasibility for repair by tetrapods.

In addition to its design advantages of steeper slopes and less material, the tetrapod structure offers an advantage in construction in that work can be carried on continuously throughout the year. The construction season for stone structures is usually limited by the weather to six or seven months. The casting and curing of the tetrapods can be carried on during periods of inclement weather. Since handling and placing of these units can be accomplished at a much faster rate than stone, the stock pile built up during the winter months can be placed without difficulty during the summer period in addition to those tetrapods manufactured during the summer.

The relative cost of a structure built of tetrapods as compared to stone may be gaged from bids obtained this year for an increment of the Crescent City breakwater. The quarries available to provide sufficient rock for work described in several of the bid schedules involved either the construction of a 4,000-foot causeway to an island or a haul of about 20 miles plus a 700-foot causeway. The bid specifying tetrapods for this work was lower than either of the foregoing by 25 percent.

In view of the many advantages apparent in this type of breakwater construction, and the depletion of rock of the size required for natural armor stone, it is believed that tetrapods or cast-concrete units of similar characteristics will have an increasing use in future breakwater construction.

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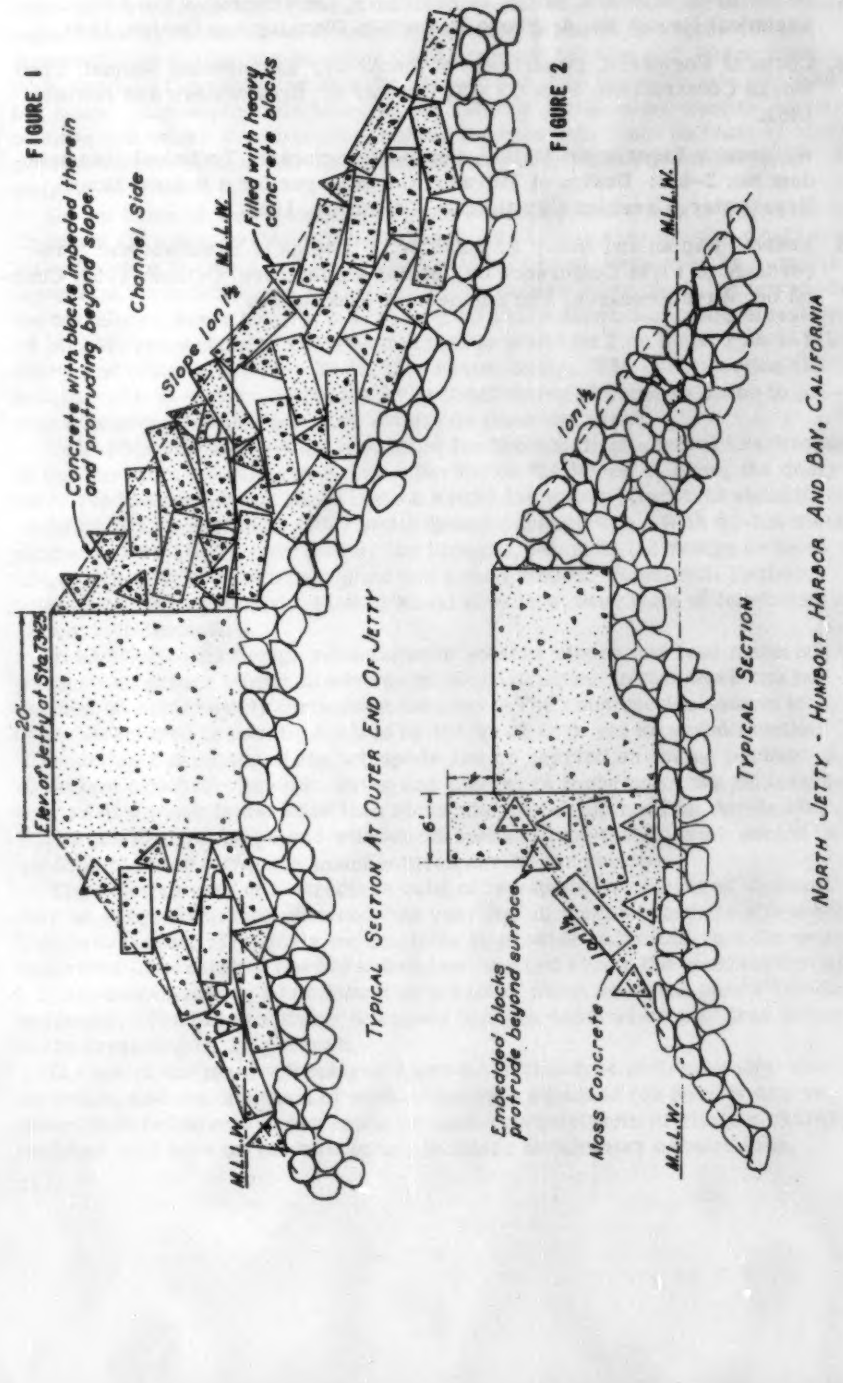
FIGURE 1
Concrete with blocks imbeded therein and protruding beyond slope.
Channel Side
Slope 1 on 1¼
M.L.L.W.
Filled with heavy concrete blocks
20'
Elev. of Jetty at Sta.73+25
M.L.L.W.
Typical Section At Outer End Of Jetty
FIGURE 2
1 on 1¼
M.L.L.W.
6'
1 on 1¼
Embedded blocks protrude beyond surface
Moss Concrete
M.L.L.W.
Typical Section
North Jetty - Humbolt Harbor And Bay, California

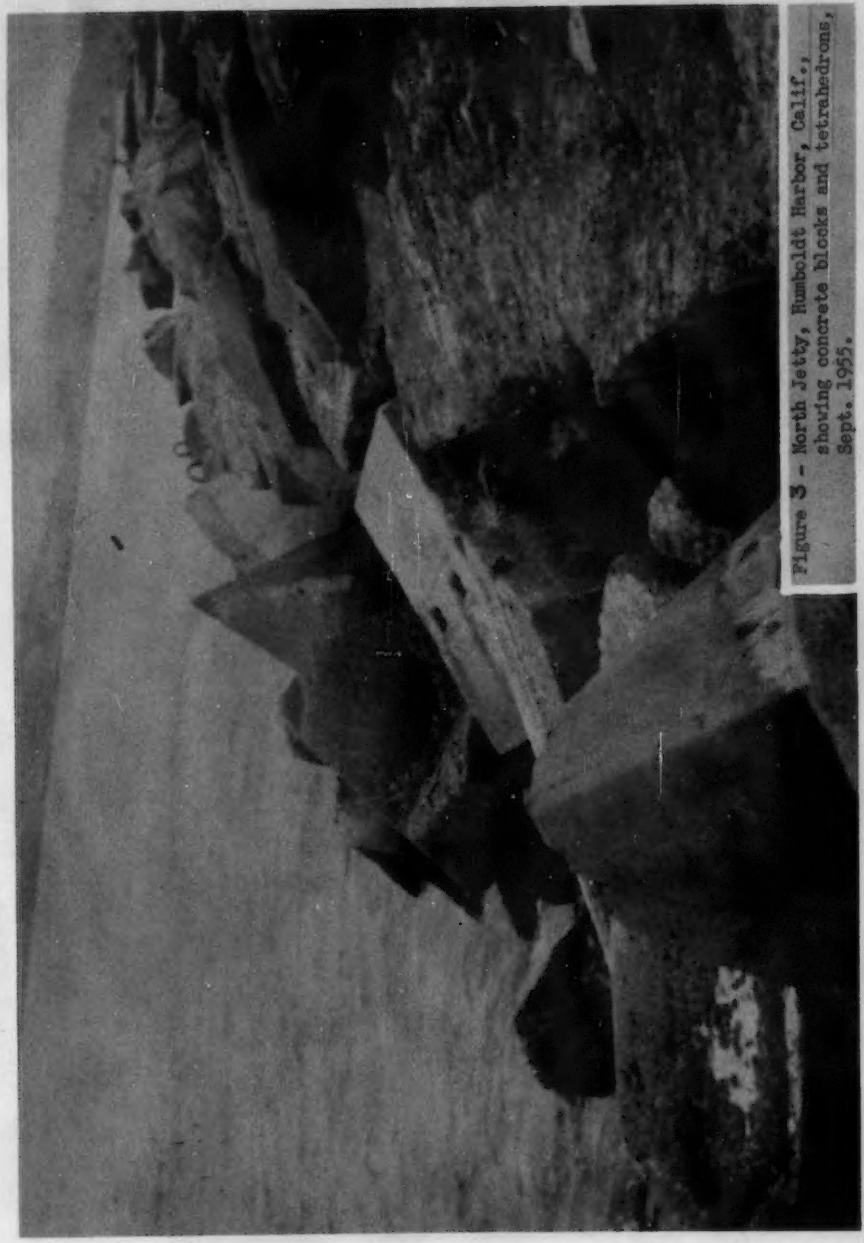

Figure 3 - North Jetty, Humboldt Harbor, Calif., showing concrete blocks and tetrahedrons, Sept. 1955.

Figure 4 - North Jetty, Humboldt Harbor, Calif., showing damaged section. Sept. 1955

Figure 5 - Crescent City Calif., 1953.
Main breakwater along left side of photograph.

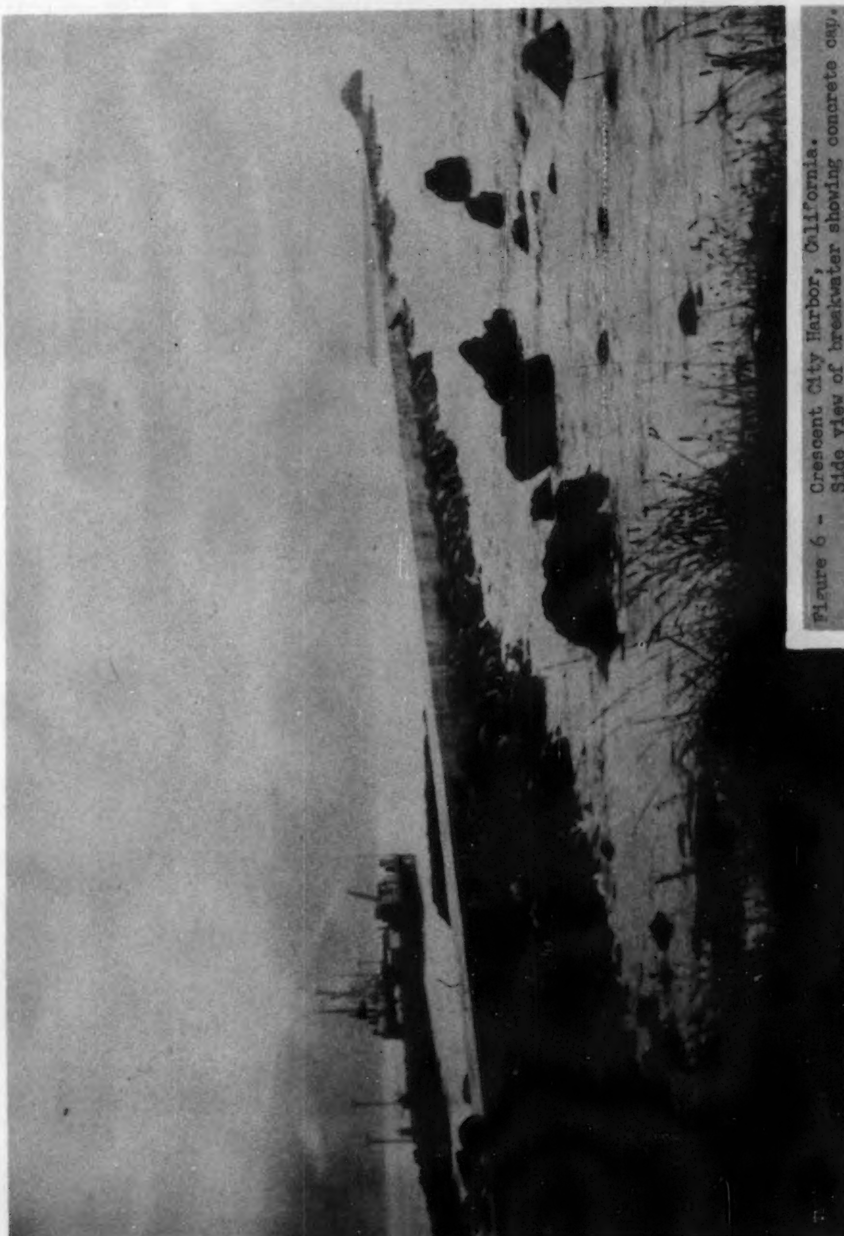

Figure 6 - Crescent City Harbor, California.
Side view of breakwater showing concrete cap.

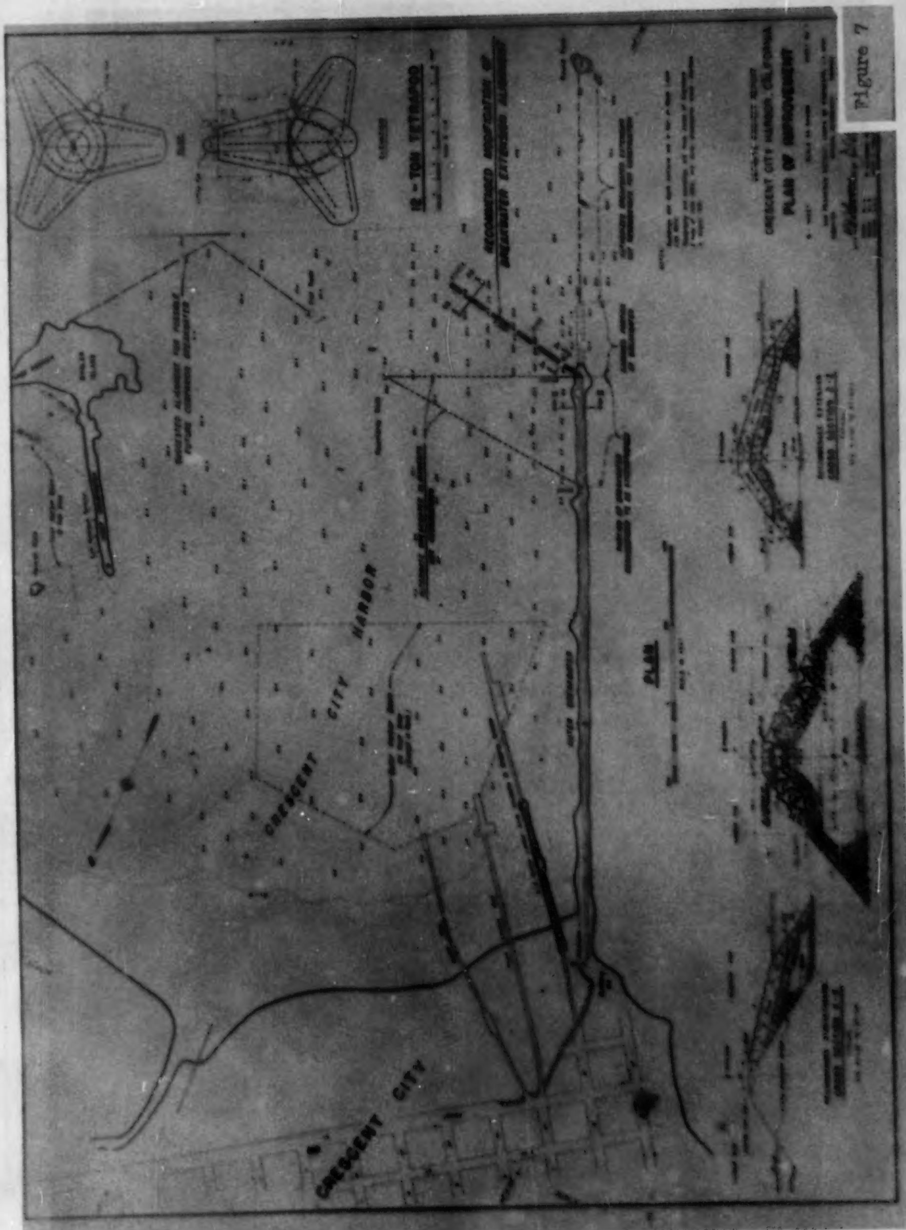

Figure 7

Figure 8 - Crescent City Harbor, California.
View along crest of breakwater.

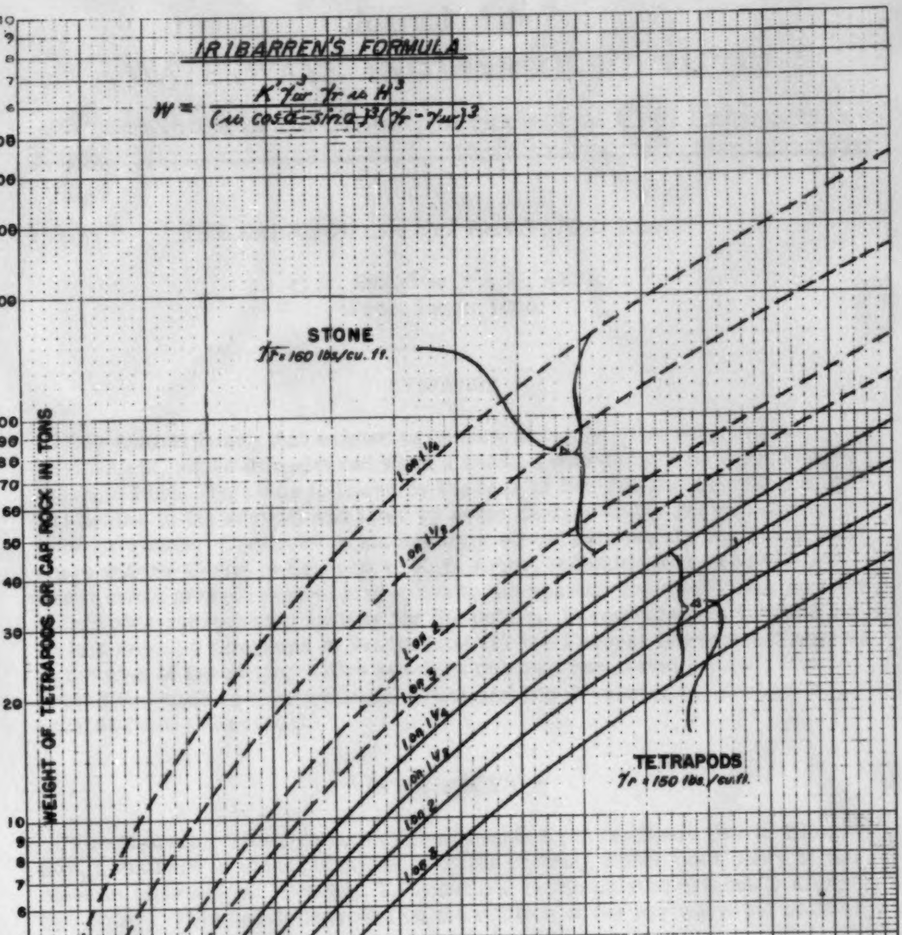
IRIBARREN'S FORMULA
$W = \frac{K' \gamma_w^3 \gamma_r \mu^3 H^3}{(\mu \cos\alpha - \sin\alpha)^3 (\gamma_r - \gamma_w)^3}$
STONE
γ_r = 160 lbs/cu. ft.
TETRAPODS
γ_r = 150 lbs./cu.ft.
1 on 1¼
1 on 1½
1 on 2
1 on 3
1 on 1¼
1 on 1½
1 on 2
1 on 3
WEIGHT OF TETRAPODS OR CAP ROCK IN TONS
HEIGHT OF DESIGN WAVE IN FEET
STONE & TETRAPOD WEIGHT
VS.
WAVE HEIGHT
IRIBARREN'S FORMULA
AVERAGE K' VALUES

FIGURE 9

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THE HARRISON COUNTY ARTIFICIAL BEACH[a]

F. F. Escoffier,* A.M. ASCE
(Proc. Paper 1060)

SYNOPSIS

The natural beach that existed originally along the Harrison County, Mississippi, shore disappeared when a seawall was completed along that shore in 1928. This was followed by the loss of backfill through the joints and drains in the seawall and later by some damage to the seawall during the 1947 hurricane. Cooperative studies carried out between the County of Harrison and the Corps of Engineers, U. S. Army, indicated that an artificial beach would provide needed protection to the seawall and would also provide a desirable recreational facility for the public. In conformance with the policy laid down by Congress in Public Law 727 the Federal Government shared in the cost of the project. The beach, which was completed in 1951, has proved its value as a recreational asset and has prevented the escape of backfill from the seawall.

INTRODUCTION

The shoreline under consideration extends from St. Louis Bay to Biloxi Bay, a distance of about 27 miles, and lies entirely within the County of Harrison in Mississippi. See Fig. 1. A natural beach existed originally along this shore. This beach depended on the erosion of the low wave-cut bluff at its landward limit for its natural supply of sand. The construction of a bulkhead or seawall to protect that bluff could therefore be expected to result eventually in the loss of the beach.

The Construction of the Seawall

In 1915 a highly-destructive hurricane destroyed over half of U. S.

Note: Discussion open until February 1, 1957. Paper 1060 is part of the copyrighted Journal of the Waterways Division of the American Society of Civil Engineers, Vol. 82, No. WW 4, September, 1956.

a. Presented before the Waterways Div., ASCE, Knoxville, Tenn.

* Chief, Hydr. Section, Mobile Dist., Corps of Engrs., U. S. Dept. of the Army, Mobile, Ala.

Highway 90 between Pass Christian and Biloxi and did about $13,000,000 worth of damage to beach-front property in Mississippi and Louisiana. The State of Mississippi then passed a law appointing a commission to study the subject and authorizing the coastal counties to issue bonds for the construction of seawalls. During the period 1925-28, Harrison County constructed about 24 miles of seawall at a cost of $3,400,000. This seawall extends from Henderson Point to the Biloxi lighthouse. It consists of an inclined concrete slab supported on piling and having its upper surface formed in a series of steps. The top elevation of the seawall varies from 8 to 11 feet above mean sea level, the height being governed by the elevation of the back-shore area. The slab is supported at its toe by a continuous concrete sheet-pile curtain wall and at the rear by square concrete bearing piles. Storm drainage is provided through the wall by means of concrete pipes which are encased in concrete collars on the seaward side of the wall.

The beach in front of the seawall disappeared a short time after the seawall was constructed and much of the sand backfill was lost through defective joints and drains in the seawall. The sidewalk then settled and disintegrated and the adjoining highway was threatened.

The Cooperative Studies

In March, 1942, the Harrison County Board of Supervisors made a formal application to the Corps of Engineers, U. S. Army, for a cooperative study to determine the best method for repairing and protecting the existing seawall against further damage, and to furnish plans for establishing and protecting a beach along the shoreline under study.

After making a study of the problem the Chief of Engineers, acting on the recommendations of the Beach Erosion Board, recommended the immediate initiation of a program of seawall repair and beach construction at an estimated initial cost of $620,000 and the institution of an adequate program of periodic inspection and maintenance of both the beach and the seawall. He further recommended that no share of the expense of the improvement be borne by the United States.

A policy of Federal assistance in the construction of works for the improvement and protection of publicly owned shores against erosion by waves and currents was set forth in Public Law 727, Seventy-ninth Congress, approved August 13, 1946. Following the enactment of this law the County of Harrison initiated in 1946 a second cooperative study to supplement the first and to reconsider the questions of Federal participation.

Before this second study could be completed the hurricane of September 1947 occurred. This hurricane moved inland over the Mississippi Delta in a northwesterly direction and passed through New Orleans. The Harrison County shore was caught in the right half of the hurricane and was exposed to winds with velocities as high as 100 mph. The wind came generally from an easterly quadrant and drove the water before it into the Mississippi Sound and the neighboring bodies of water. The result was the creation of a storm tide that increased in height progressing from east to west and reached a maximum known stage of 15.2 msl at Bay St. Louis, Mississippi.

No observations are available in regard to the height of waves in the Sound during the hurricane. However, formulas for the height of waves in shallow bodies of water, developed from observations made in Lake Okeechobee,

Florida, indicate that the height of waves at the seawall probably did not exceed 10 feet.

The seawall was completely overtopped and much of the land behind the seawall was inundated. The waves passed over the seawall to attack the highway and those of the buildings that were not located on high ground. Over 900 buildings were destroyed and over 8500 were damaged. The cost of repair to roads and bridges in Mississippi amounted to $544,000. Most of the fill behind the seawall was lost but the seawall itself failed at only a few points. Failure, where it did occur, was caused by hydrostatic pressure which lifted the stepped slab from its supporting piling.

The second cooperative study was completed in 1948. This study took into account the changes caused by the 1947 hurricane and went into greater detail than the first study, particularly in regard to repairs to the drainage system. It, like the previous one, indicated that a hydraulic fill placed seaward of the seawall, if properly maintained, would be the most suitable means of stopping the leakage of sand through the seawall and of prolonging the life thereof. Such a beach would, in addition, provide a recreational facility for the general public. In conformance with the policy set forth in the new law, the Chief of Engineers, again acting on the recommendations of the Beach Erosion Board, recommended Federal participation to the extent of $1,133,000 toward the repair of the seawall and its protection by the construction of a beach, this amount being one-third of the original cost of the seawall, which was $3,400,000. The amount of Federal aid was based on the seawall proviso of Public Law 727 which states ". . . that where a political subdivision has heretofore erected a seawall to prevent erosion, by waves and currents, to a public highway considered by the Chief of Engineers sufficiently important to justify protection, Federal contribution toward the repair of such wall and the protection thereof by the building of an artificial beach is authorized at not to exceed one-third of the original cost of such wall . . . ."

The plan of improvement formulated in the studies was adopted in the 1948 River and Harbor Act. This plan included repairing of the stepped concrete slab by the pressure concrete or "gunite" method, replacing backfill, pumping the hydraulic fill for the beach, and reconstructing the drainage system. The total volume of sand required for the beach was 5,985,000 cubic yards. The drainage plan included a collecting sewer back of the seawall, discharging through relatively few laterals across the beach. All drainage lines were designed to minimize differential settlement and to assure tight joints in the interest of preventing the infiltration of sand. Sewer pipes laid across the beach were anchored at their seaward ends by means of creosoted timber piling structures. At the larger outfalls the plan specified that drainage be carried across the beach between two parallel rows of interlocking concrete sheet piles.

The Construction of the Beach

Before the dredging operation was started the county built three rather long groins of broken concrete. These three groins, together with the Gulfport and Pass Christian harbor peninsulas and a few previously existing concrete groins at the Biloxi Lighthouse, divide the beach into five separate compartments. The most important of the groins is the one at Henderson Point which confines the beach at its western extremity.

The dredging operation was carried out by two hydraulic dredges working toward each other, one beginning at Henderson Point, and the other at the Biloxi Lighthouse. Work was commenced early in January 1951, and completed in December of that year.

The sand was obtained from borrow pits at least 1500 feet from the seawall and from either side of the Gulfport harbor. First a ridge about 1000 feet long at a distance of about 300 feet from the seawall was constructed. The intervening space between the ridge and the seawall was then filled with an amount of material somewhat in excess of that required to bring the beach to the specified cross section. The excess was provided to forestall any need for the dredges to return for repumping and also to provide material for refilling behind the seawall. This process was then repeated in steps of about 1000 feet. It was subsequently found that the retaining dike in beach construction is unnecessary. Actually, the dike prevented the runoff of the undesirable fine material.

To provide continuous drainage from the land side of the seawall, depressions were left across the beach at the locations of the outfalls. The placing of the outfalls followed the hydraulic fill as closely as possible. The flow line of the discharge end of the pipes was established at—0.5 msl at distances from the seawall varying from 248 feet for the 42-inch pipes to 300 feet for the 18-inch pipes. The upper half of the outer end of the pipes was left exposed but most of the remainder of the pipes was entirely covered with sand.

The specified beach cross section included a 50-foot level crown at elevations 5 msl adjacent to the seawall and a seaward slope of 1 on 50 down to natural ground surface. Every effort was made to place the material within the prescribed slope limits, but, as the underwater slope assumed by the sand was much steeper than that specified, it was necessary to overpump the outer slope and later to dress the excess material landward by means of bulldozers. In this connection, it should be pointed out that the present practice is to place the estimated quantity with the berm at the required elevation, and to allow the slope seaward of the berm to assume its natural slope under wave action.

As soon as the beach was in place it was observed that the waves were producing a weak littoral drift toward the west. That the drift is westward could be inferred from the accumulation of sand on the east side of the groins and outfalls and from the erosion that has appeared on the west of these structures. Sand has escaped around the outer end of the Henderson Point groin and has formed a small pocket beach on the west side thereof. This can be seen in Fig. 2.

A ridge or berm having an average crest elevation of about 3 feet msl was formed. This change in the shape of the beach left many of the outfall lines protruding out into the water. It also left pools of water impounded landward of the ridge. To eliminate the pools and to obtain a more uniform beach slope the sand was regraded with bulldozers to a new cross section made up of a 1 on 100 slope extending from the seawall to the ridge crest and a 1 on 10 slope from the crest to the low-water line. Seaward of the low-water line the sand assumed a rather flat natural slope.

In some cases the erosion on the west side of the pipes was severe enough to be objectionable. To remedy this the county engineers developed a type of spur groin that is proving effective. These spur groins are built of broken concrete and extend westward from the outer end of the pipes and parallel to the seawall. They are about 50 feet in length and are placed to the same

elevations as the top of the pipes. The spur groins function by intercepting the waves and thereby reducing the attack on the beach behind them.

The outfall pipes and flumes tend to fill with sand at their outer ends. However, in most of the pipes the storm waters flowing through the pipes are capable of flushing out the sand. The same is not true of the flumes and these have to be cleaned out from time to time. The large outfall flume at the Veterans Hospital at Gulfport has been a particular problem in this regard. This problem has been partially solved by extending the east wall about 50 feet with a broken-concrete groin to impound the littoral drift. The results have been good but it is believed that the groin should be extended somewhat farther and raised to a height of about 3 feet. A groin of the type previously described, to reduce erosion to the west of the flume, has been constructed at the outer end of the west wall.

The observed slope adjustments, the effect of the sand ridge on the outfalls, and other experiences with the finished beach were used as a guide when the County extended the beach eastward along the waterfront in Biloxi. This extension, which was completed in 1954, was designed with an elevation of 5 msl at the seawall, a 1 on 100 slope for a distance of 220 feet to a crest at el. 2.8, and then a slope of 1 on 10 to the natural floor in the Sound. The pumping of the beach was successfully accomplished without the use of a retaining dike at the seaward edge of the beach, and the use of such dikes in the future construction of artificial beaches is considered inadvisable. No appreciable slope adjustment due to the action of the tides and waves has been observed.

Comparison of surveys over a long period of time is the only accurate means of determining the rate at which sand is lost from a beach. An accurate estimate of this rate for the Harrison County beach cannot as yet be given. In an attempt to determine the changes that had taken place between 1951 and 1953 groups of 5 cross sections each were taken at about one mile intervals. They were so located as to avoid coming under the groin effects of the pipe outfalls. A typical section is shown in Fig. 3. The readings at the toe of the 1951 slopes are indefinite because of the mud found at that point. Allowance had to be made in the calculations for the sand that was removed from the beach to backfill the seawalls. It was finally estimated that the annual loss from the beach extending from Henderson Point to the Biloxi Lighthouse was roughly 32,500 cubic yards for that period.

Maintenance

The problem of maintenance has been discussed by MacArthur.[1] The maintenance program is carried out by a crew of about 50 men. The work includes reshaping the beach; filling low areas; removing wind-blown sand from the seawall and the adjacent roadway; removing grass, debris, and trash from the beach; cleaning outfall pipes and flumes; and constructing broken-concrete groins as needed at the seaward ends of these pipes and flumes. The equipment used includes 6 trucks, 8 bulldozers, 2 Seaman mixers, one small dragline, and one sanitizer. The Seaman mixers are used to remove grass and other vegetation from the beach. This they do by completely uprooting the vegetation and leaving it on the beach to be dried by the sun. Debris left on

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1. MacArthur, Arthur. Maintenance of the Harrison County, Mississippi, sloping beach, to be published in "Shore and Beach."

the beach by the tide is gathered by hand and burned. The sanitizer is a tractor-drawn machine that picks up the sand to a depth of about four inches and removes all sharp-edged shells, broken glass, and other objects that would otherwise be a menace to the users of the beach. The cost of maintaining the beach averages about $10,000 per month, according to the engineers employed by the county. No beach replenishment by dredging has been necessary since the completion of the fill and no major repairs to the drainage system or to the seawall have been required.

Results

The artificial beach along the Harrison County seawall has proved effective in preventing the escape of backfill. Too short a time has elapsed since its completion to determine its resistance to erosion, particularly in view of the fact that no hurricane or severe tropical disturbance with accompanying high tide and severe wave action has directly attacked the project area. Observations to date indicate that the rate of erosion has been small and that replenishment will only be required at rare intervals.

The beach is used extensively as a recreational facility. Although no actual count has been taken of the number of daily visitors to the beach, its extensive use has been noted by county officials and by representatives of the Corps of Engineers. Also a considerable expansion in hotels, tourist cottages, and other tourist accommodations has been noted.

ACKNOWLEDGMENT

The writer wishes to express his appreciation to Mr. Arthur MacArthur, County Engineer, and to Mr. J. K. Muether, Assistant County Engineer, Harrison County, for furnishing much of the data presented in this paper.


HARRISON COUNTY
JACKSON COUNTY
Ocean Springs
DEER ISLAND
BILOXI
LIGHTHOUSE
BILOXI CHANNEL 10' x 150'
SEAWALL
Broken-concrete groin
Edgewater Park
Mississippi City
Biloxi River
L. & N. R.R.
GULFPORT
GULFPORT SHIP CHANNEL 30' x 220'
Broken-concrete groin
Long Beach
U.S. 90
SEAWALL
Wolf River
Pass Christian
Broken-concrete groin
Henderson Pt.
ST. LOUIS BAY
Bay St. Louis
MISSISSIPPI SOUND
INTRACOASTAL WATERWAY 12' x 150'
GULF
CAT ISLAND
SHIP ISLAND
SHIP ISLAND BAR CHANNEL 32' x 300'
GULF OF MEXICO
SCALE IN MILES
0 1 2 3 4 5
VICINITY MAP
LOCATION OF STUDY
ST. TAMMANY
HANCOCK
HARRISON
JACKSON
MOBILE
NEW ORLEANS
N

FIG. 1

FIG. 2

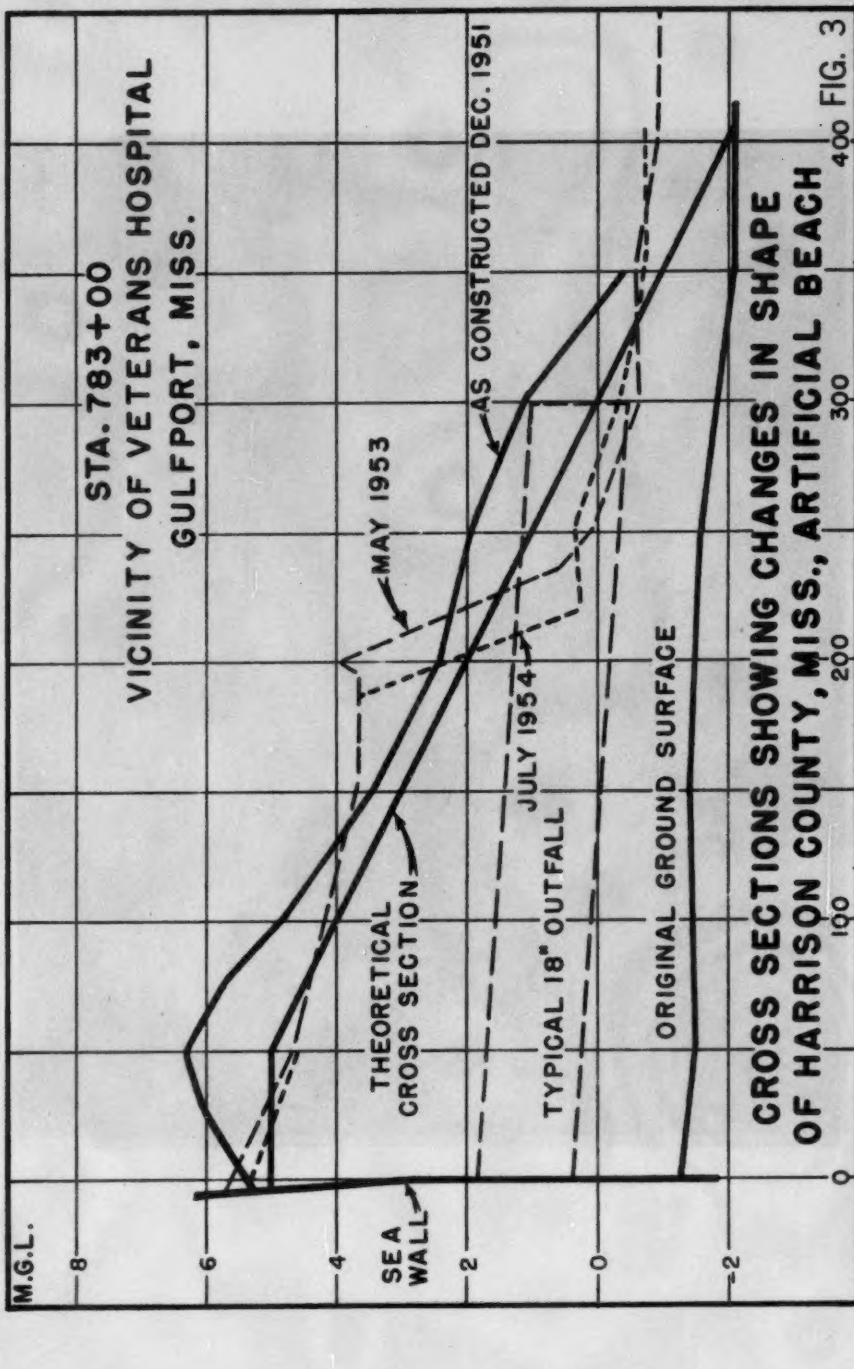


FIG. 3

CROSS SECTIONS SHOWING CHANGES IN SHAPE OF HARRISON COUNTY, MISS., ARTIFICIAL BEACH

EXHIBIT A

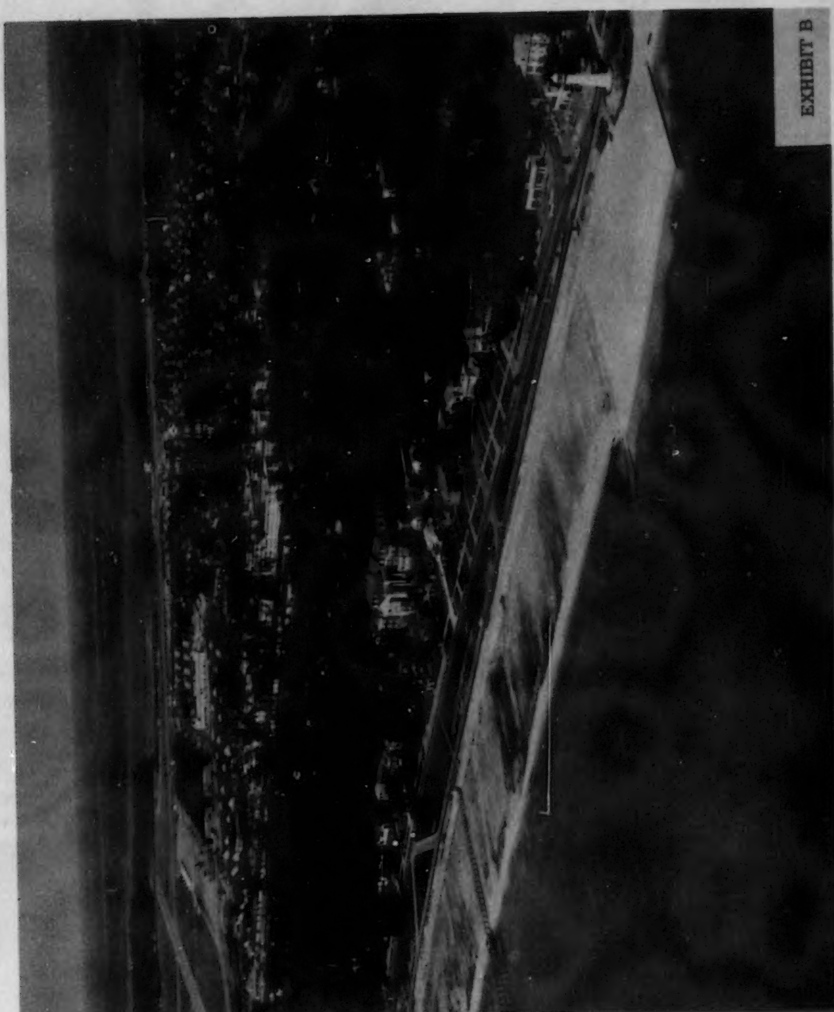


EXHIBIT B

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

GROWTH OF COMMERCE: TENNESSEE AND CUMBERLAND RIVERS[a]

G. M. Dorland,[1] M. ASCE, and G. R. Bethurum, Jr.[2]
(Proc. Paper 1061)

ABSTRACT

The United States inland waterways system carried over 173 billion ton-miles of freight last year, an increase of 101 percent in the past quarter century. Tennessee and Cumberland Rivers growth in commerce has greatly exceeded the national average. This paper attempts to present the reasons for this growth.

SYNOPSIS

As the growth of internal commerce keeps pace with America's industrial expansion, waterways carry an increasingly larger share of the Nation's freight movements. The Tennessee and Cumberland Rivers are important arms in the more than 28,000 mile inland waterways system. The traffic gains on these two rivers have been phenomenal.

History

All of the great civilizations of recorded history have been centered around mass transportation by water. Egyptian civilization was essentially the civilization of the Nile River and after establishment of cities in the valleys of the Nile and Euphrates a considerable system of water transportation developed. There was early evidence, too, of man's effort to develop waterborne commerce with the landlocked areas. Eannadu, a Euphratean King, circa B. C. 4000 built brick-lined canals for transportation purposes and the

Note: Discussion open until February 1, 1957. Paper 1061 is part of the copyrighted Journal of the Waterways Division of the American Society of Civil Engineers, Vol. 82, No. WW 4, September, 1956.

a. Presented before the Waterways and Harbors Div., ASCE, Knoxville, Tenn., June 5, 1956.

1. Dist. Engr., Nashville Dist., Corps of Engrs., U. S. Dept. of the Army, Nashville, Tenn.

2. Chief, Management Branch, Nashville Dist., Corps of Engrs., U. S. Dept. of the Army, Nashville, Tenn.

great Hammurabi, circa B. C. 2000 complained that a navigable canal connecting Erech with the Euphrates was blocked so that ships could not go up it. The Roman Empire maintained its unity, prosperity, and strength over many centuries by virtue of mass transportation on the Mediterranean Sea—transportation of grain, olive oil, and of products of the handicraft industries—without which the empire could not have subsisted.

Throughout the entire history of Western civilization, navigable waterways have provided the avenues for cummunity growth. The first colonists on the Atlantic Seaboard found their way inland over the many coastal rivers—The St. Lawrence, Hudson, Delaware, Potomac and James. They established our principal eastern cities at points easily accessible to both the tide water and the inland rivers. The construction of canals for the improvement of navigation began almost as soon as the Nation was founded. As America grew beyond the Seaboard, two great natural waterways systems proved to be of vital importance. One was the St. Lawrence—Great Lakes System, over which explorers and traders passed into the headwaters of the Mississippi. The other was the Mississippi, which with its great tributary system, the Ohio and its feeder streams the Monongahela, the Tennessee and the Cumberland, provided a water route to the vast territory lying just west of the Appalachians.

Probably the earliest carrying place of any magnitude between the Atlantic Plains and the Mississippi was the portage route from the Yadkin to the headwaters of the Watauga and the French Broad. Over it passed Daniel Boone, Robertson and other adventurous spirits to establish the Watauga Settlements and furnish a base of operations for two great streams of emigrants to the west, one of which passed down the Tennessee River.

As early as 1821 Congress saw the wisdom of Federal assistance in waterways development. By the outbreak of the Civil War the Federal Government had given some four and one-half million acres of land to the states to help in canal building. The importance of navigation on the Tennessee River had received national recognition long before the Civil War, however. In 1824, John C. Calhoun, then Secretary of War, recommended its improvement to President Monroe as part of the broad program of waterway development. In 1828, Congress authorized surveys for a canal and locks around Muscle Shoals and in 1831 construction was begun by the State of Alabama with funds realized from the sale of 400,000 acres of land donated by the United States for that purpose. A lateral canal with 17 locks, constructed throughout the length of Muscle Shoals, was opened to navigation in 1890.

Recognition of the importance of the Cumberland River as a highway for commerce also received early attention. In 1830, an appropriation of $60,000 was made by the State of Tennessee for those rivers lying west of the Cumberland Mountains and east of the Tennessee River. The first survey, under an Act of Congress, was made in 1832 when $30,000 was appropriated for surveys of the Cumberland River. In response to a resolution of Congress and after an additional appropriation of $30,000 had been made available, in 1834 a report was submitted on the "Improvement of the Cumberland River for the navigation of it by Steam Boats."

The history of the development and utilization of our waterways is marked by three distinct periods: First, the development of the rivers for seasonal use of the keelboats, flatboats and rafts and later for the fabulous packet boats; secondly, the period of decline; and, thirdly, the rebirth of the waterways system as a mass transportation artery.

The turn of the century saw the end of the first period. Utilization of the

Tennessee and Cumberland Rivers followed the national pattern and after reaching a peak in commerce of approximately 1,800,000 tons on the Tennessee and 700,000 tons on the Cumberland, commerce for the subsequent twenty-two years showed a marked decline.

Prior to the turn of the century the advantages of railway transportation had made themselves apparent, and the National Government had begun its aid to railroads as it had the waterways. While the Corps of Engineers continued vigorously to expand and improve navigation on the Great Lakes, on the rivers of the eastern slopes, and upon the great Mississippi and her tributaries, the Corps of Topographical Engineers explored the west, discovering and laying out rail routes. Loans, land grants and other assistance were also bestowed upon the young and growing railroads by the Federal Government and by the states and local governments.

Faster, more flexible, and more competitively aggressive than the early steamboat lines with their limited capacity over relatively unimproved rivers, the railroads for a time virtually ended navigation on most of our inland waterways.

Period of Transition

With the great economic changes in our Nation and the rapid trends toward industrialization, by World War I it became increasingly apparent that no region could be self-sufficient and at the same time maintain an adequate level of productivity to achieve a high standard of living. It became clear that we had to specialize. The availability of certain natural resources favors one region as a center of coal and steel production, another of grain production, a third of manufacturing processes. The isolated region which must be self-sufficient in a wide diversity of products suffers from low productivity. It is inevitably poor, sparsely populated and culturally stagnant. For this reason, nearly all our great cities and communities have grown on regional specialization along the routes of navigable water transportation and herein lies the vital character of the system of waterways. The revival of water transportation is still a new and growing thing. It began only a generation ago, and its beginnings were slow. It is only since World War II that the curves of tonnage and ton-mileage have swooped upward to a significant place in the national transportation picture.

As recently as 1943 inland waterways carried only two and one-half percent of the Nation's freight, while railways carried 72 percent. By 1953 inland waterways and the Great Lakes carried more than 16-1/2 percent, almost the equal of the 17 percent carried by motor vehicles, while the railroads' share dropped to 52-1/2 percent. This does not mean, however, that the increase in waterways movements was at the expense of the railroad. Because of industrial expansion, railway freight movements on a ton-mile basis increased 67 percent during the period 1940 to 1950 while national income from railroad operations increased 163 percent over the same period.[1]

Waterways, railways, highways and airways are all indispensable segments of the national transportation system, and the national interest requires all to be in a healthy and thriving condition. For every headlined occasion when railway and waterway interests conflict, there are dozens of unnoticed

1. Source: Statistical Abstract of the United States, 1955.

occasions where their interests coincide, complement and mutually reinforce each other. It has been demonstrated time and again—at Houston, at Kansas City, at Louisville and Cincinnati, along the Gulf—that water transportation tremendously stimulates industry in the regions it serves, and that railways share abundantly in the new business thus built up. The fact that there is no essential conflict between railroad and waterways is proved by the recent statement of Leo K. Nielson, President, Tennessee Central Railroad. He says, "all transportation media are being utilized in America's phenomenal industrial expansion. We must encourage new developments for our area even though a part of the transportation operations may be in conflict with our personal objectives. Experience has proven that new industry and economic growth means new business for everyone." Further evidence of the benefits from a cooperative operational program is contained in a recent statement by C. M. Roddewig, President of the Chicago and Eastern Illinois Railroad. In citing superior coal handling facilities developed in recent years, Mr. Roddewig specifically singled out the barge loading facilities at Mount Vernon, Indiana, for shipping Illinois and Indiana coal to power plants and industrial users in Kentucky, and another installation for unloading barges of Kentucky and West Virginia coal destined for steel mills in Chicago and Northern Indiana.[1]

Revival of Water Transportation

A number of factors have contributed to this revival of water transportation.

First, the improvement of our inland waterways system for deep-draft navigation and the development of adequate navigation locks to accommodate modern equipment.

Second, the construction of flood control and multipurpose projects to provide year-around regulated flow in the navigable waterways.

Third, the adaptation of radar and ship-to-ship and ship-to-shore radio to inland waterways use. These developments have completely revolutionized river shipping and have greatly reduced delays on account of unfavorable weather and operational adjustments of the locks.

Fourth, but equally important, the development of the twin-screw diesel powered towboat. The development and great expansion of river transportation in recent years rests on the obvious principle that it is easier to push a load than it is to carry it. The optimum of transportation economy is one which maximizes the ratio of load to power—the ratio of tonnage carried to the power unit necessary to move that tonnage from one point to another. The use of the river system represents the ultimate exploitation of this economy ratio. By way of contrast, the ratio of load to power is very low in highway trucking and, in consequence, highway trucking represents a relatively expensive traffic movement. Trucking commerce, however, is growing tremendously; but the great economy of the high ratio of load to power requires extremely large units of movements and, in consequence, this favorable ratio can be realized only when massive tonnages of commodities are transported in single moving units.

Where rapid and frequent movements of relatively small units are

1. Source: Waterways Journal, February 4, 1956.

required, obviously trucking is ideal; but where sufficient volumes can be brought together for the necessary large units of waterway movement, this waterway movement represents the maximum ratio of load to power and achieves greatest savings. The twin-screw diesel towboat is a direct application of these principles of economy. (See Exhibits A and B).

While the great volume of river commerce has been low-cost bulk commodities, there is a general misconception that waterways movement is economical only for this type of shipments. Low-cost transportation is economically sound for all commodities, and water transportation can be utilized for any commodity—high-cost or low-cost—which can be assembled in sufficient tonnages to make up an economical tow. For example, shipments of electrical apparatus, engines, machine tools, construction equipment, aircraft and aircraft parts, and automobiles, all appear in waterways statistics for recent years.

Another factor in the tremendous growth of river commerce is the improvement of terminal facilities and the mechanization of loading and unloading operations. Terminal facilities are the gateways to our waterways, and the increase in number of these gateways and the reduction of the money, time and effort required to pass through them have been vital in bringing more freight to the water's edge.

Tennessee and Cumberland River Commerce

Commerce on the Tennessee and Cumberland Rivers has shown a phenomenal growth during the past decade. (See Exhibit C) On the Tennessee River, with its deep-draft channel and modernized facilities, commerce grew from 2,399,000 tons in 1946 to 9,760,000 tons in 1955, according to preliminary statistics, representing a 307 percent increase. On the Cumberland River traffic growth was from 1,027,000 tons to 2,900,000 tons over the same period, representing a 182 percent increase. Probably more significant, however, was the increase of traffic on a ton-mile basis. On the Tennessee River, commerce increased from 192,805,241 ton-miles in 1946 to 1,500,000,000 ton-miles in 1955. On the Cumberland River during the same period, the ton-mile increase was from 115,708,937 to 329,708,000. During this same period, total waterborne commerce of the United States increased approximately 49 percent.[1]

To adequately handle this expanding volume of commerce, there are now 88 commercial terminals in operation on the Tennessee River and 29 on the Cumberland River.

As can be seen on Exhibit D, principal commodities carried during 1955 on the Tennessee River were coal and coke, sand and gravel, petroleum products, grain, and iron and steel products. Chemicals, forest products, and automobiles also moved in substantial quantities. Of the 26 different commodity groups moved on the river, 16 moved in greater volume than ever before. Carbon electrodes, liquid chlorine, and newsprint moved for the first time on the Tennessee.

On the Cumberland River, as indicated on Exhibit E, stone, sand, and

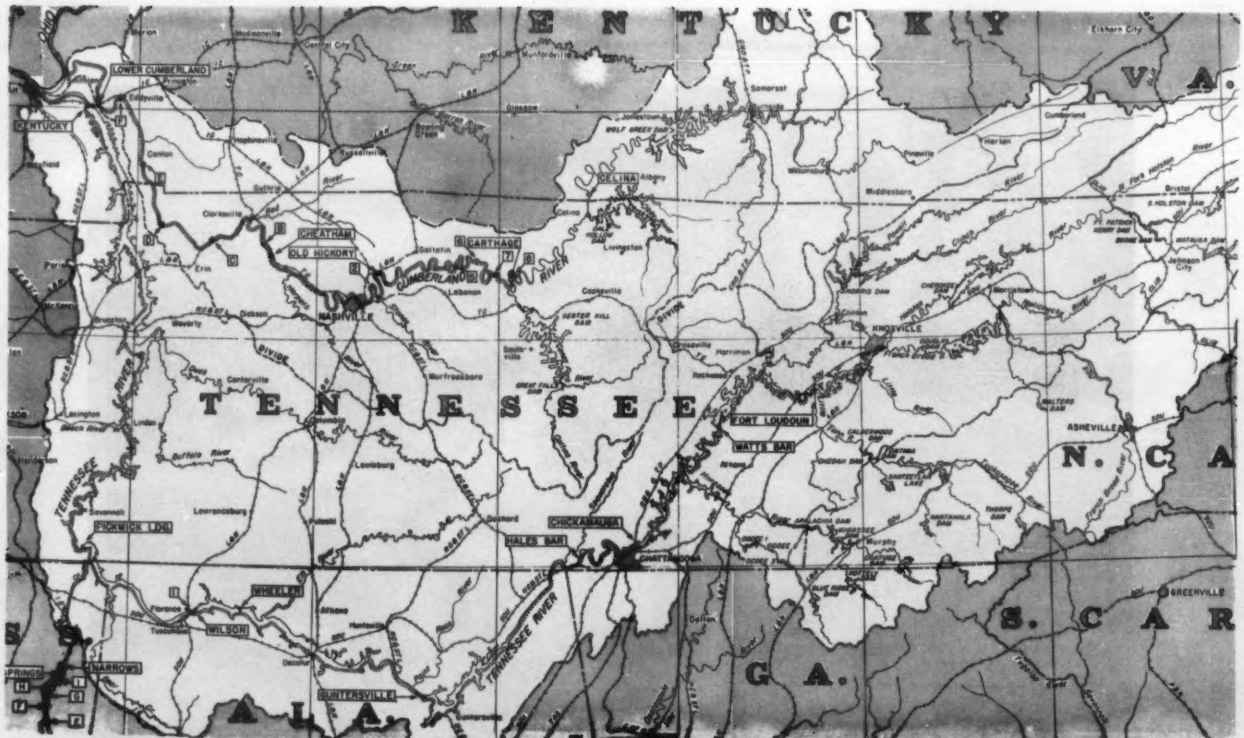
1. Source: Annual Report of the Chief of Engineers for 1946 and 1954 commerce projected on the basis of American Waterways Operators' estimate of increase in traffic for 1955 over 1954.

Towboat ROBIN, Arrow Transportation Company, upbound on the Tennessee River with a 13-barge, 20,517 ton, mixed tow of grain, steel pipe, construction steel, asphalt and coal.

EXHIBIT A

The Motor Vessel VIRGINIA of the Cumberland River Sand and Gravel Company pushes upriver to Nashville with a mixed commodities tow of 10,750 tons, 960 feet in length.

EXHIBIT B

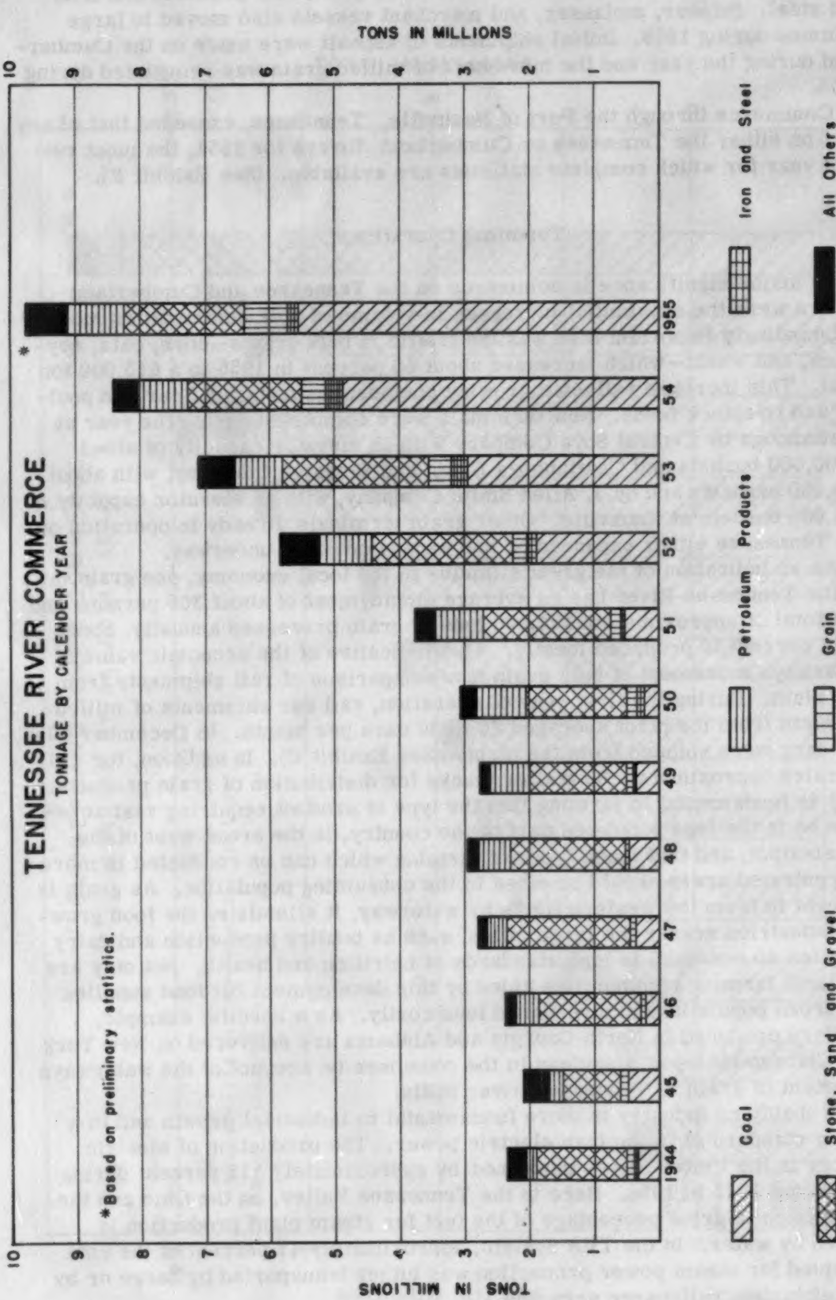


TENNESSEE AND CUMBERLAND RIVERS

An improved channel of nine-foot depth is available on the Tennessee River from its mouth to Knoxville, Tennessee. Existing project depth on the Cumberland River is six feet from the mouth to Carthage, Tennessee. Projects currently under construction will provide an improved nine-foot channel over this same section.

EXHIBIT C

EXHIBIT D


TENNESSEE RIVER COMMERCE
TONNAGE BY CALENDER YEAR
TONS IN MILLIONS
1
2
3
4
5
6
7
8
9
10
1944
45
46
47
48
49
50
51
52
53
54
1955
Coal
Stone, Sand and Gravel
Petroleum Products
Grain
Iron and Steel
All Others
*Based on preliminary statistics.

gravel led all commodity groups, followed by petroleum products and iron and steel. Sulphur, molasses, and merchant vessels also moved in large volumes during 1955. Initial shipments of asphalt were made on the Cumberland during the year and the movement of milled grain was reinitiated during 1955.

Commerce through the Port of Nashville, Tennessee, exceeded that of any port on either the Tennessee or Cumberland Rivers for 1954, the most recent year for which complete statistics are available. (See Exhibit F).

Terminal Operations

Of major significance in commerce on the Tennessee and Cumberland Rivers were the substantial increases in volume of bulk commodities moved. Outstandingly important also was the traffic in bulk grains—corn, oats, soybeans, and wheat—which increased about 60 percent in 1955 to a 615,000 ton total. This increase reflected growing southern markets for flour and poultry and livestock feeds. New terminals were completed during the year at Chattanooga by Central Soya Company with an elevator capacity of about 3,000,000 bushels and Chattanooga Grain and Elevator Company, with about 200,000 capacity and by J. Allen Smith Company, with an elevator capacity of 103,000 bushels at Knoxville. Other grain terminals already in operation on the Tennessee either expanded or had expansion plans underway.

As an indication of the great stimulus to the local economy, one grain mill on the Tennessee River has an average employment of about 305 persons and of a total of approximately 174,000 tons of grain processed annually, about eight percent is produced locally. Also indicative of the economic value of waterways movement of bulk grain is a comparison of rail shipments from this plant. During the first year of operation, rail car shipments of milled products from the plant averaged 20 to 30 cars per month. In December 1955, 305 cars were shipped from the plant. (See Exhibit G). In addition, the mill operates approximately 80 trailer trucks for distribution of grain products.

It is fundamental to farming that the type of product requiring vast acreages be in the less populated part of the country, in the areas west of the Mississippi, and that those types of farming which can be conducted in more concentrated areas should be close to the consuming population. As grain is brought in from the western fields by waterway, it stimulates the food growing industries nearer the large cities, such as poultry production and dairy supplies so essential to high standards of nutrition and health. Not only are the local farming communities aided by this development but food supplies for urban populations are rendered less costly. As a specific example, broilers produced in North Georgia and Alabama are delivered on New York and Chicago tables at a savings to the consumer on account of the waterways shipment of grain to Tennessee River mills.

Probably no industry is more fundamental to industrial growth and to a rising standard of living than electric power. The production of electric energy in the United States increased by approximately 112 percent during the period 1945 to 1954. Here in the Tennessee Valley, on the Ohio and the Mississippi a great percentage of the fuel for steam plant production is moved by water. In the TVA System, approximately 47 percent of the coal received for steam power production was either transported by barge or by a combination rail-barge shipment.[1]

1. Source: Tennessee Valley Authority Annual Report—1955.

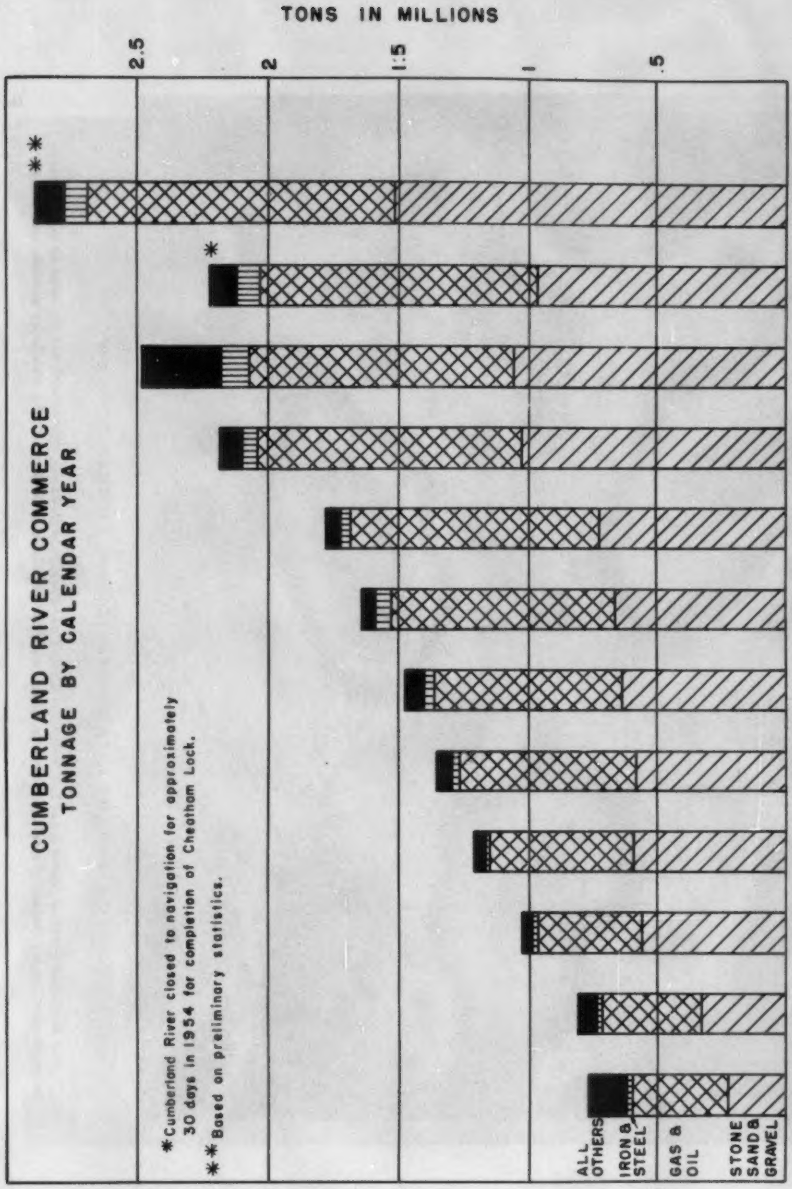
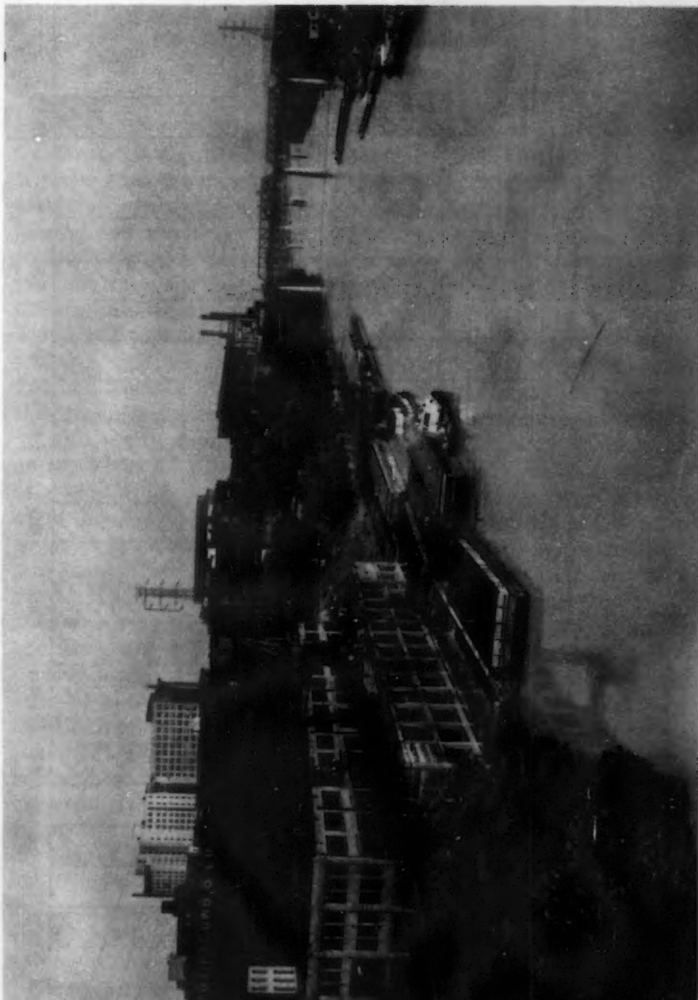

CUMBERLAND RIVER COMMERCE
TONNAGE BY CALENDAR YEAR
TONS IN MILLIONS
2.5
2
1.5
1
.5
*Cumberland River closed to navigation for approximately 30 days in 1954 for completion of Cheatham Lock.
**Based on preliminary statistics.
ALL OTHERS
IRON & STEEL
GAS & OIL
STONE SAND & GRAVEL
1944
45
46
47
48
49
50
51
52
53
54
1955
*
**

EXHIBIT E



CUMBERLAND RIVER MUNICIPAL TERMINAL, NASHVILLE, TENNESSEE

The Municipal Terminal, under lease to Cumberland Storage and Warehouse Company, offers facilities for barge to rail and barge to truck interchange. Barges of steel mill products, sugar, and flour are shown moored around the warehouse awaiting unloading.

EXHIBIT F

EXHIBIT G

An outstanding example of waterways movement of high-value manufactured products is the large volume of automobiles currently being moved on the Tennessee River. At least one specially designed towboat for automobile commerce is in almost continuous operation on the Ohio and Tennessee in shipment of vehicles from Louisville, Evansville and Cincinnati to Guntersville, Alabama. (See Exhibit H). Tonnage on automobiles for 1954 totalled 43,328.

In addition to the direct savings from waterways movements of bulk commodities the indirect savings is apparent from an analysis of a combined water-truck coal shipment operation from the West Kentucky fields to a Georgia Power Company steam plant. Shortly after initiation of a combined waterways movement of coal from Uniontown, Kentucky, to Guntersville, Alabama, and truck shipment from Guntersville to Rome, Georgia, a rail rate reduction of $1.25 per ton from mine to plant was issued to meet the competition.

SUMMARY

The Mississippi River is the backbone of an improved inland waterways system unmatched in the world. (See Exhibit I) At the South it opens onto the 1,100 mile long Gulf Intracoastal Waterway, feeding the chemical, petroleum, metal, and other industries of Louisiana and Texas. Improved on the basis of a 10,000,000 ton annual commerce, the Gulf Intracoastal Waterway in 1954 carried 37,000,000 tons—almost four times the estimate of the previous decade. Above the Gulf, the lower Mississippi River up to Cairo, Illinois, taps the rich agriculture of the South and carries some 25,000,000 tons of varied commerce. Midway of its course, the river opens westward into the Missouri River waterway, now in the latter stages of construction, to serve a huge grain and livestock region which is only now beginning to develop its vast productive capacity as its water and power resources are developed. Eastward, it reaches into the great industrial valley of the Ohio, where, due in large part to the advantages offered by the waterway, 2,500 new industries representing an aggregate investment of about $10,000,000,000 have already arisen since the end of World War II.

Meanwhile, at the eastern end of the Great Lakes, construction is under way on the St. Lawrence Seaway, an improvement which promises to inaugurate a new cycle of expansion in the Lakes Region and will likely parallel that of the Gulf Area. Not only will it for the first time permit modern seagoing vessels, requiring depths up to 27 feet, to enter the Great Lakes, but it will tap the strategic open-pit iron mines of Labrador and play a vital role in enhancing the position of our heavy manufacturing industry—particularly in the Great Lakes Basin.

Thus, when developments now under way are completed, the people of America will have one integrated system of water transportation reaching from the Mexican Border to the shores of Labrador, and from the Appalachians far into the western plains. Here in the Cumberland and Tennessee River Basins we are strategically located to receive maximum benefits from this well integrated system.

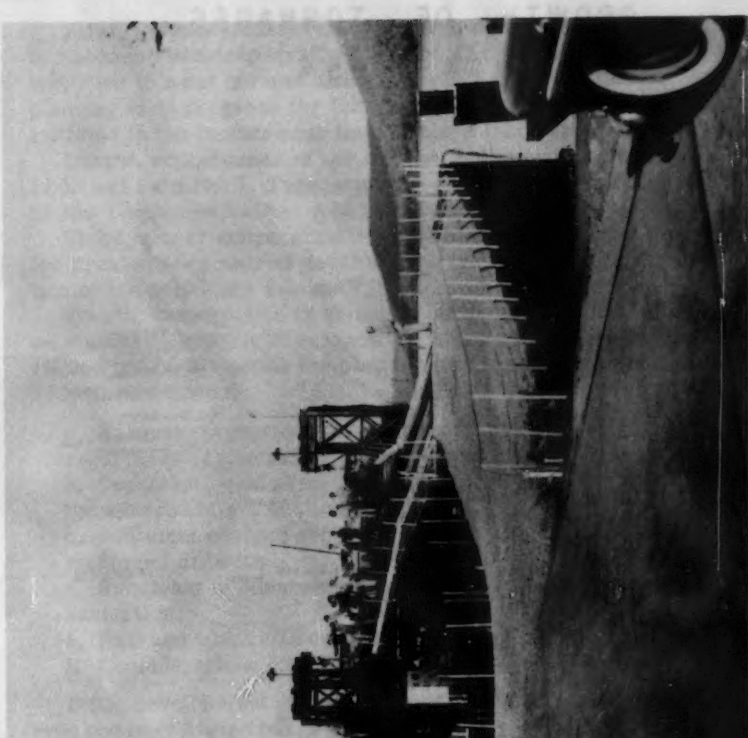


EXHIBIT H

Automobile Unloading Terminal, Guntersville, Alabama, under lease to Commercial Barge Lines, Inc. Movable ramp in center is counterbalanced and motor driven for adjustment to operating level of automobile barge.

MISSISSIPPI RIVER SYSTEM
GROWTH OF TONNAGES
1944—1954

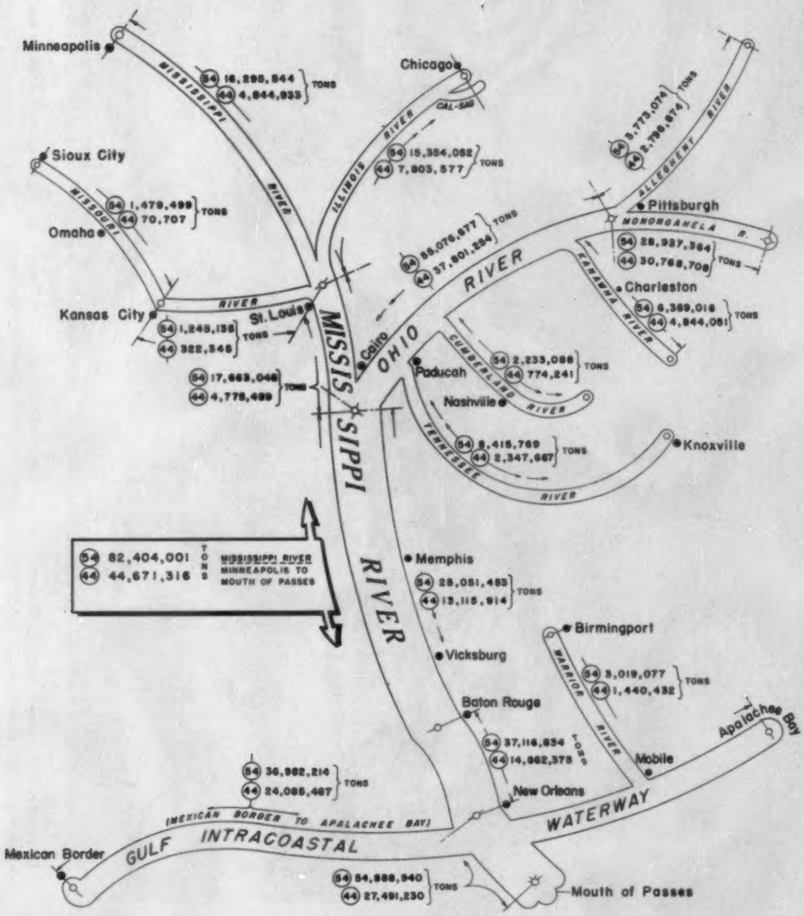


EXHIBIT I

CONCLUSION

For full realization of the economic advantages which can be made available to the Cumberland and Tennessee Area our objective should be five-fold:

First, the modernization of the lower section of the Cumberland River to accommodate deep-draft navigation and to provide adequate lockage facilities to meet current and foreseeable traffic demands. Advanced planning is in progress for this improvement and construction could be initiated in the forthcoming fiscal year if funds are made available.[1]

Second, replacement of the existing bottleneck at Muscle Shoals and Lock and Dam No. 1, Tennessee River. Planning is currently under way by the Tennessee Valley Authority for design of new navigation facilities.[2]

Third, closer cooperation between all transportation interests to work for area-wide industrial development and expansion to assure full utilization of the potentials available to this area.

Fourth, construction of commercial terminal facilities should be based on standards which will enhance the loading and unloading operations and reduce the layover time for barges and tows to a minimum. These standards should include:

a. Adequate provisions for continuous loading or unloading operations regardless of fluctuation of pool levels.
b. Construction of adequate harbor or breakwater facilities to minimize the adverse affect of wind action.
c. Sufficient channel depth at terminals to permit full operations at maximum drawdown.
d. Provision of adequate mooring facilities to eliminate hazards to navigation.
e. Rail and truck connections.
f. Considerations for future expansion.

Fifth, development of two-way commodity shipments to replace the current one-way loaded barge shipment and empty non-revenue return. The savings in a two-way revenue shipment will provide further economic advantages to waterway movements.

1. See paper entitled "Barkley Project, Cumberland River" presented by A. E. Dykes at Waterways Division, ASCE Convention, Knoxville, Tennessee, June 1956.
2. See paper entitled "Design Considerations for the New Lock at Wilson Dam on the Tennessee River" presented by R. A. Monroe at Waterways Division, ASCE Convention, Knoxville, Tennessee, June 1956.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

LOWER CUMBERLAND PROJECT: KENTUCKY AND TENNESSEE[a,b]

Albert E. Dykes*
(Proc. Paper 1062)

SYNOPSIS

The Lower Cumberland Project, located near the mouth of the Cumberland River, is a key unit of the eleven reservoir system for the utilization of the water resources of the basin in the interests of flood control, navigation, hydroelectric power, and allied purposes. The dam will be at mile 30.5, and will be some 7,985 feet in length, composed of a 110 x 800-foot lock, a spillway surmounted by 20 crest gates, a power plant of 130,000 kilowatts installed capacity, switchyard, and earth embankment closure sections. A canal some 2.5 miles above the dam will connect the reservoir with the Kentucky Reservoir on the Tennessee River, and will permit interchange of waterborne commerce between the two river systems as well as a two-way diversion of flows for flood control and hydroelectric power production. Operating levels of the two projects will be identical.

The reservoir will be 118.2 miles in length, extending upstream to the Cheatham navigation and power project, and will cover some 96,000 acres at the maximum controlled level. Rather extensive relocations of existing highways and railroads will be involved, together with the removal of portions of a number of towns and rural communities. A large wildlife refuge in the lower reaches also will be affected.

The entire project is currently estimated to cost $167,000,000. Detailed foundation investigations are well along and other necessary design studies are under way, with a view toward initiation of actual construction in 1957.

Note: Discussion open until February 1, 1957. Paper 1062 is part of the copyrighted Journal of the Waterways Division of the American Society of Civil Engineers, Vol. 82, No. WW 4, September, 1956.

a. Presented before The Waterways and Harbors Div., ASCE, Knoxville, Tenn., June 5, 1956.

b. Under the provisions of Public Law 537, 84th Congress, 2nd Session, Chapter 320, the Lower Cumberland Dam and Reservoir are designated Barkley Dam and Lake Barkley, respectively.

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INTRODUCTION

The Lower Cumberland Project, under consideration for many years by the Corps of Engineers, is now being readied for construction. Much discussion has taken place through the intervening period since its inception in 1939, some favorable, some adverse. Changing economic conditions, nationwide recognition of the needs for full development of the water resources potentialities of our streams, unprecedented floods, phenomenal growth of electric power requirements, integrated improvement of the inland waterways system, and public acceptance, even demand, of the personal advantages to be derived from large impoundments—all have contributed towards the inclusion of the project in the current Federal public works program. The problems to be overcome are many and complex, but none are insoluble, and eventually we may expect to enjoy the benefits from this monumental multi-purpose development in the lowermost reaches of the Cumberland River Basin which will provide for the needs of modern navigation, control floods, produce electric power, and embrace other collateral uses such as water supply, irrigation, conservation, and recreation.

Turning to the physical aspects, the Lower Cumberland Dam site is on the Cumberland River at mile 30.5 (above mcuth) in Lyon and Livingston Counties, Kentucky, near Grand Rivers, Kentucky, and approximately 160 river miles below Nashville, Tennessee. The reservoir will extend 118.2 miles upstream to Cheatham Lock and Dam, nearing completion at river mile 148.7, near Ashland City, Tennessee; and will lie within Livingston, Lyon, Caldwell, and Trigg Counties of Kentucky, and Stewart, Montgomery, Houston, Cheatham, and Dickson Counties of Tennessee. Also included in the plan is a canal connecting Lower Cumberland Reservoir with Kentucky Reservoir on the Tennessee River. The canal will be located in a saddle through the narrow ridge separating the Tennessee and Cumberland Rivers at a point about 2.5 miles upstream from the dam site.

The plan of improvement comprises the construction of a multiple-purpose earth and concrete dam, navigation lock, canal, and hydroelectric power generating plant, the principal features of which are contained in Table 1. Plate 1 shows the project location with respect to the other elements in the Cumberland River Basin plan, while plate 2 shows the layout and selected details of the structures. Plate 3 is an artist's perspective of how the dam will look upon completion. (Discussion of the basin plan is covered in the paper presented at the Knoxville convention entitled, "The Comprehensive Development of the Cumberland River and Tributaries," by F. P. Gaines and J. T. Dennison.)

The data submitted in the following paragraphs were developed in connection with the authorizing document (Senate Document No. 81, 83d Congress, 2d Session) and from limited subsequent studies, and may be adjusted to some degree depending upon the outcome of the detailed planning studies being made prior to actual construction. The current proposals are sufficiently firm, however, to indicate the basic structural features and to determine reliable project economics.

Legislative Background

As previously noted, studies as to the feasibility of the Lower Cumberland

Project were initiated in the late 1930's in connection with a review of the old "308" report on the Cumberland River Basin with a view to developing an over-all comprehensive plan for the utilization of the water resources of the river and its major tributaries. A report on the investigations was submitted in 1941 envisioning the construction of the Lower Cumberland Project as a part of the plan. World War II intervened, however, and during those trying days the report lay dormant while our material and manpower resources were directed towards preserving the integrity of our country. Upon cessation of hostilities, water resources development plans were resumed on a nation-wide basis and the Nashville District revised and resubmitted the Cumberland Basin report, again recommending construction of the Lower Cumberland Project. As an alternate to the so-called "high dam" plan, the feasibility of a "low dam" plan, consisting of two navigation only projects, Kuttawa (Eureka) at mile 32.2 and Dover at mile 87.6, was presented. In reviewing the report, the Board of Engineers for Rivers and Harbors and the Chief of Engineers recommended the alternate navigation only projects. This decision was reached on the basis of additional public hearings for affected parties, opposition by the then-incumbent governors of Kentucky and Tennessee to the high dam scheme, and the apparent lack of justification for additional power at that time. The Congress accepted the recommendations of the Chief of Engineers and, in the Rivers and Harbors Act of 1946, authorized the plan for the Cumberland River which included the two navigation only dams.

During the intervening period following that Act, a marked change took place in economic conditions and in public opinion. As a result, the Corps of Engineers was directed by the Senate Committee on Public Works on 31 July 1951 to restudy the basin plan as contained in House Document No. 761, 79th Congress (the basis for the 1946 authorization) with a view towards substituting the Lower Cumberland Project for the two low head navigation dams. The ensuing report, prepared by the Board of Engineers for Rivers and Harbors in collaboration with the Nashville District, was submitted to the Congress and published as Senate Document No. 81, 83d Congress. On the basis of the recommendations contained therein, and after receiving other testimony, the Lower Cumberland Project was inserted by the Congressional Committees in the 1954 Rivers and Harbors Act and authorized for construction. Two specific provisos were included: (1) an initial monetary limitation equivalent to the estimated cost of the two low dams and (2) a specific requirement that replacement be provided for fish and wildlife losses caused by the construction. The project can be initiated within the monetary limit, although an increase will be required later as construction progresses. The fish and wildlife aspects are covered later in this paper.

Site Selection

Numerous geologic problems confronted the District in the selection of an adequate site for the Lower Cumberland Dam. An early reconnaissance had disclosed that two of the major problems would be the avoidance of the major faulting known to be present in the lower reaches of the Cumberland River and the location of a site with a rock bench of sufficient width to accommodate the concrete structures.

Geologic mapping performed by the Kentucky and Tennessee State Geology Departments helped to outline the general areas of major faulting. Many

likely sites below mile 20 were eliminated because of the presence of these faults, and by the existence of unfavorable rim conditions, which would be very costly to seal against leakage. Several sites between mile 20 and mile 48 were tentatively selected that avoided the known faults and met other engineering requirements. The next and more expensive step was to determine which of these sites met the foundation requirements.

Since bedrock along the lower Cumberland River was known to be deeply buried except where the river impinged on the valley walls, use of the Shepard refraction-type seismograph was considered the most economical method of locating the bench, with core drilling used to confirm and develop the most desirable sites. The seismograph program involved a series of lines along the axes of possible sites in the 28-mile reach, resulting in the location of only one area near mile 30 where rock was of sufficient lateral extent to accommodate the required structures. The conditions indicated by the seismograph were tentatively confirmed by probings and the site at mile 30.5 was selected. It is worthy of note that the seismic method, although not always fully reliable, proved to be an exceptionally valuable and economic tool in the Lower Cumberland program, and the results were fully verified by subsequent work using conventional drilling methods.

In connection with the exploration program, particular attention was paid to the possibility of a site at about mile 48.5 in order to eliminate the serious relocation problems at Kuttawa and Eddyville and to railroads and highways. The site was, of necessity, discarded, however, due to faulting and depth of rock, and to the high cost of providing a navigable channel to that point. The connecting canal would also be much longer through one of the upstream tributary divides and would be prohibitively expensive.

In the fall of 1944, funds were made available for additional seismic work and for a limited amount of core drilling to extend and substantiate the previous information on the selected site. The information derived from this program further indicated that the mile 30.5 site offered the best all around location for the project. Ten of the holes drilled in the flood plain at this site had 100 percent core recovery; and in the area which was investigated under the masonry structures, no extensive weathered zone was indicated.

Following project authorization and upon the receipt of planning funds in August 1955, detailed foundation investigations were initiated at the site. Since no subsurface investigations had been made in the river section prior to this time and in order to take advantage of the low water season, the drilling program was begun in the river. The vertical holes drilled early in this program disclosed that the bedrock contained numerous enlarged, high angle joints, particularly in the downstream area of the structure. The drilling pattern was, therefore, extended somewhat, and it was found that the conditions improved for several hundred feet upstream from the original layout. Accordingly, the axis was shifted to take advantage of the better rock. The holes had generally outlined the weathered zones but did not provide the detail required since it appeared that they failed to intercept many of the nearly vertical joints. The remaining detailed exploration in the river, therefore, was performed by means of 45 degree angle holes which were generally drilled along the line of the structures. The drilling pattern was designed so that the holes overlapped each other vertically in order to intercept solution channels. This method of exploration along with information obtained from the vertical drill holes and a detailed systematic probing program provided the information required to outline the major weaknesses in the foundation. The river explorations were completed at the end of January 1956.

In late October 1955, detailed explorations were commenced in the right bank flood plain. After the dam site was shifted, angle hole work was also adopted on the banks. Information obtained from the holes drilled in the powerhouse and spillway areas on the right bank indicate that the solution is not as severe as found in the river; however, this assumption is based on only a few holes and must be proven by further work.

Only one hole has been drilled on the left bank (lock side) in the current program and only a few widely scattered holes in the 1944 program. These holes do not provide enough information on which to accurately determine the severity of solution work in this area, but there is some indication from the solution pattern found in the river that trouble may be encountered in the vicinity of the lower miter sill of the lock.

It became necessary to suspend field operations during February and March 1956 because of high flows in the Cumberland, Ohio, and Tennessee River watersheds which held the lower Cumberland River at or near flood stage. The explorations are again under way and are proceeding on an expedited schedule.

There is little doubt that the development of the foundation of this dam site will be a complex problem. Detailed drilling similar to that performed in the river section is required under the masonry sections on the banks, along the canal, and under the embankment, before the foundation can be completely designed and any necessary sealing requirements determined. Even with this necessary detailed drilling, very rigid inspection and supervision will be required during construction in the development of the final foundation lines.

Operating Levels

The Cumberland and Tennessee River Basins are intimately related by physiography, meteorological conditions, and cultural and economic characteristics. Both streams have their headwaters in the Appalachian Mountain ranges on the east, follow a somewhat crescent-shaped path southwestward, then turn and flow north or northwestward to enter the Ohio River only 12 miles apart. The Cumberland Basin nests or "spoons" within the concavity of the Tennessee Basin. Through a quirk in geologic fate in ages past, the two streams were not joined near their lower ends, but were separated by a restraining barrier and permitted to flow generally parallel for their last 50 miles or so. At the narrowest point the dividing ridge between the natural streams is only about two or three miles in width. The Tennessee River has been impounded by the Kentucky Dam just below this point. The Lower Cumberland Dam site is practically due east of Kentucky Dam and, upon its impoundment, the reservoirs of the two projects will be only about a mile apart. The practicability, therefore, of joining the two projects through the construction of an interconnecting canal is immediately apparent. Such a connection will allow coordinated operation for the fullest utilization of the regulated flows of both streams for hydroelectric power production, and interchange of vessels between the two waterways. In this last feature, sailing distances between the two streams are reduced over 60 miles, while downbound traffic to the lower Ohio can transit Kentucky lock at a saving of about 20 miles. Incidentally, each lock could function as an alternate to the other in case of outages or disaster which will eliminate the necessity of an auxiliary lock at either location.

By virtue of the interconnection, it is logical that the operating levels of the Lower Cumberland project be made to correspond with those of Kentucky. Accordingly, the storage capacity for control of floods at Lower Cumberland has been established between elevation 354 and elevation 375 during the winter, or flood season, months (the pattern of floods is well established in this region through a long history). The storage capacity during this period will be 1,555,000 acre-feet. When passing large flows through the reservoir, a considerable amount of storage is utilized under the backwater curve. In order to compensate for this loss when storage is not required for control of floods on the Ohio and Mississippi Rivers, provision will be made for drawing down the reservoir to as low as elevation 346. At the end of the flood season, the pool will be raised from elevation 354 to elevation 359 in order to increase power production.

It is not to be inferred from the preceding statements that operating procedures have been fully developed. The problems to be faced are unique, to say the least. In fact, the author's inquiry into the situation did not disclose a similar multi-purpose development in the United States, although diversion of flows between watersheds for single purposes is rather commonplace. It is impossible to integrate two basins such as the Tennessee, with a drainage area of 40,910 square miles, and the Cumberland with 17,720 square miles, without facing problems which will tax the ingenuity of the best engineering talent. Long and continuous study will be necessary to satisfy all of the various requirements, both prior to construction and after the projects are in operation. The final sizing of the canal will be governed by its ability to accommodate power flows and storage interchanges without causing excessive velocities. An efficient forecasting system of inflows to both reservoirs is essential so that proper balancing of the pools can be obtained through outflow regulation. Differences of a few tenths of a foot can be tolerated without difficulty since excessive velocities will not be created by this slight head differential.

Structural Features

The dam site is in an alluvium filled valley of considerable width, flanked by rather low-lying hills. The river at this point flows close to the west or left abutment with the natural flood plain extending to the east or right abutment. Rock is quite deep under most of the flood plain but rises to within about 90 feet of the surface near the right river bank, extends across the river at a depth of about 20 feet, and then rises to form the left ridge. In view of this, the most economical design (see plate 2) is a combination rolled-earth fill embankment across the flood plain and concrete structures at the river channel. The right embankment will be approximately 5,800 feet in length and will be constructed to elevation 385.0, or a maximum height of about 50 feet above the present ground level. After a short non-overflow section 188 feet in length, a concrete power plant, containing five 26,000 kw units, will occupy the next 470 feet and will extend almost to the natural right river bank. Usable material excavated for this structure and from the tailrace will be placed in the embankment and used to construct an adjoining switchyard. A spillway some 852 feet in length with 20 gate openings and a crest at elevation 330 (current studies indicate the number of gates may be reduced and the crest lowered 5 feet) will occupy the present river channel and extend slightly

into both banks. A gantry deck will be constructed at elevation 395 across the power plant and spillway structures. The concrete lock will be placed within the narrow left flood plain and will require about 201 feet of space. The clear inside dimensions of the lock will be the same as the upstream Cheatham Dam, or 110 feet wide by 800 feet long, permitting passage, without breakage, of anticipated standard tows in a single lockage. The top of the walls has tentatively been set at elevation 382.0. The sills will provide a minimum of 14 feet clearance for vessels using the 9-foot waterway. A short tie-in embankment 474 feet long will then provide closure with the left abutment. The deepest excavation is expected under the power plant where the bottom of the draft tubes is at elevation 250. Owing to the existence of a recently completed bridge on U. S. Highway No. 641 only 0.5 mile downstream, there will be no necessity for a vehicular bridge across the dam. It is planned, however, to provide a railroad bridge across the face of the dam as a part of the relocation of the Illinois Central Railroad which will be inundated as a result of the project. The lock will be extended upstream into the pool to permit the railroad to cross the lower approach, thereby eliminating the expense of installing and operating a movable section to provide clearance for waterborne traffic.

As envisioned in the preliminary studies, the interconnecting canal will be excavated through the ridge with a width of about 600 feet and a bottom at elevation 330. The length of excavation will be about 1.5 miles although the heaviest portion of the work will be confined to about one mile. A highway bridge will be required to maintain access along Kentucky Highway No. 453. In the original planning it was considered, in view of the possible disparity in flow characteristics, that regulating control gates might be required across the canal. The cost of this feature was included in the authorizing legislation in order that the work could be performed if essential, recognizing that the detailed hydrologic studies might result in its elimination and a substantial saving obtained.

Relocations

Unlike many of the headwater reservoir areas, the Lower Cumberland pool will occupy lands that have been traversed for many years by a rather complete highway and railroad system. The adjustments required to restore the system to equivalent serviceability will be quite extensive as well as expensive. Large scale topographic maps and many detailed surveys will be necessary to determine the full extent of the reservoir effects and the remedial measures to be taken. Based on current examinations, however, it is indicated that it will be necessary to construct or raise portions of State and Federal highways totalling 45 miles, and numerous sections of county roads totaling 65 miles, where they cross arms of or lie within the limits of the reservoir. Adjustments to numerous farm and miscellaneous roads also will be required to provide reasonable access to affected areas. The only railroad relocation, as such, involves a 11.5-mile length of track of the Illinois Central Railroad in the lower portion of the reservoir; however, at various separated sections throughout a total length of about 18 miles on the Tennessee Central Railroad, it will be necessary to protect the roadbed against wave action. Considerable stone revetment will be needed to insure the stability of fills. In like manner, protective measures will be required at

intervals along a similar 18-mile length of the Louisville and Nashville Railroad. Present estimates indicate that the highway and railroad relocation costs will approximate $24,000,000. Utility adjustments and cemetery removals, always a sensitive item, are expected to be relatively minor in nature, costing in the range of 3/4 million dollars.

Land Acquisition

One of the most difficult and controversial features of a project of this nature and magnitude is the matter of acquisition of the right to use private property. Individuals are displaced, often without their volition, for a project whose benefits generally accrue to others. In the early stages when Lower Cumberland was first being discussed, the opposition of local residents was particularly vehement. At the present time, although still being expressed, this opposition appears to have abated. Several factors are believed responsible. Civic groups, realizing the inherent advantages both locally and nationally, have sponsored the development. Additional incomes derived from recreation and public usage have also played a part. Fair and equitable payments for lands acquired are expected, resolving an often-expressed fear that lands would be taken without proper compensation.

The general policy of the Corps of Engineers in acquiring property has undergone some change in recent years as a result of experience gained on many civil works projects throughout the country and as a result of inter-agency studies. The present policy as followed by the Corps and the Department of the Interior in projects of this nature is in accordance with a joint agreement adopted in October 1954. Under this agreement, land permanently flooded will be acquired in fee, as will all land in the fluctuation or flood storage zone up to a 5-year frequency under project conditions, on the theory that the area could not be utilized for any useful purpose due to the risks involved. From the 5-year frequency to the maximum anticipated levels only an easement to flood will generally be acquired. Flowage easements ordinarily will be taken in the upper reaches of the reservoir where lands have always been subject to frequent flooding. The policy will permit limited taking in fee to the upper limits at strategic points where areas are necessary for public access or for operational purposes. The Fish and Wildlike requirements in this specific case also may dictate additional fee areas to meet the proviso of the enabling legislation.

Including severance, it is presently estimated that some 104,650 acres of land will have to be acquired for project purposes. Usable agricultural land, either cultivated or subject to being cultivated, comprises 74,650 acres of that amount, the remainder being in woodlands or otherwise not adaptable to cropping practices. A preliminary gross appraisal indicates that some $24,000,000 will be involved in the purchase program.

A number of communities will be affected directly by the impoundment. The situation will be critical at Kuttawa and Eddyville, Kentucky, while at Cadiz and Canton, Kentucky, and Dover, Cumberland City, and Clarksville, Tennessee, some adjustments may be required. The first two towns, located in the lower reaches of the reservoir, will have a large portion of their commercial and residential properties flooded, and they are faced with the problem of either losing their identities or relocating above the reservoir limits.

The Corps of Engineers is not vested with specific authority to relocate

towns or portions thereof within project areas, authority being based only upon a right to acquire lands for project purposes at fair market values as determined by qualified and competent appraisers. The relocation of a city or portion thereof is entirely a matter of choice with municipal officials and affected land owners, and the municipality must formulate plans of its own to relocate. The responsibility for the selection of a new site and the acquisition thereof rests with the municipality. The Government's obligation in such matters is limited to replacement of streets, water, sewers, power lines, telephone lines, and other utilities, with facilities of equal utility in the relocation site. The Government bears the entire cost of replacement in kind; however, if the local officials elect to incorporate betterments, such additional costs must be borne by the municipality. Where a municipality or a portion thereof is abandoned and there are no plans for relocation, the Government pays no compensation for streets, utilities, etc. As to whether there is in fact a relocation of a municipality is a matter of degree, and definite rules in the determination of the Government's obligation cannot be set forth. The principal elements for consideration in reaching a decision are (1) that the majority of the property owners within the affected area have indicated that they will move to the new site; (2) a definite assurance on the part of municipal officials, in the form of a resolution or ordinance, that the town will be relocated to a new site; and (3) that lands for the relocation site will be acquired and sponsored by local interests.

The Kentucky Woodlands National Wildlife Refuge, under the jurisdiction of the U. S. Fish and Wildlife Service and comprising some 65,600 acres between the Cumberland and Tennessee Rivers from the vicinity of Eddyville, Kentucky, to U. S. Highway No. 68, is within the effects of the Lower Cumberland Reservoir. The Refuge is used for the propagation of deer, turkeys, and small game and provides protected nesting and rest areas for migratory waterfowl. The impoundment will influence the wildlife resources therein, as well as in other areas outside of the refuge boundaries. A preliminary report by the U. S. Fish and Wildlife Service indicated that the net results would be detrimental to the existing resources and recommended that necessary replacement facilities be provided through acquisition in fee of additional lands contiguous to the refuge and through other adjustments. At the present time, the Service is initiating a survey of the area, with matching funds contributed by the Corps, in order to define the problem and to indicate remedial measures required.

One other special feature is worth noting. On the Cumberland River at Dover, Tennessee, the Fort Donelson National Military Park, maintained by the National Park Service, commemorates a decisive Federal victory in the War Between the States. Some of the lower river batteries and possibly an access road will be affected by the Lower Cumberland Reservoir. Some corrective measures may be required to preserve this installation in its present condition.

Project Economics

The Lower Cumberland Project will be costly—in fact, at the present estimate of $167,000,000, it will represent a greater outlay of funds than for any other single project east of the Mississippi River. Physically, however, it is not the largest, and its cost probably would be exceeded if certain other completed structures could be related to today's prices. The major cost items

that will be involved are:

Cofferdams	$8,920,000
Lock	23,171,000
Dam	24,104,000
Power Plant	36,004,000
Canal	11,501,000
Lands	28,370,000
Relocations	26,310,000
Clearing and Miscellaneous	8,620,000
Total Estimate (July 1955 Base)	167,000,000

Allocated according to the separable costs-remaining benefits method, as generally recommended for use by Federal agencies, power would bear $67,828,000 of the total costs, flood control $55,064,000, and navigation $44,108,000.

The annual charges as currently estimated, including interest and amortization on the investment, replacement of certain items, an allowance for taxes foregone, and operation and maintenance are $7,714,000. This amount includes $884,000 for the item of taxes foregone.

The benefits, likewise reduced to an annual basis to provide a direct comparison with the charges, are substantial. Construction of new navigation facilities is amply justified by the savings that will be realized on existing traffic and on the new traffic expected to develop as a result of provision of a modernized 9-foot channel on the Cumberland River connecting with the Ohio and Mississippi River systems of similar project depth. In addition, savings will accrue due to greatly reduced operation, maintenance and replacement costs for existing obsolescent navigation facilities in this reach of the river. Indicative of the demand for use of the Cumberland River for navigation purposes is the continuing increase in traffic volume experienced during recent years in spite of the inadequate existing facilities. The provision of high dam facilities in conjunction with the canal to Kentucky Reservoir also makes alternative routes available for traffic between either the Cumberland or Tennessee Rivers and the Ohio River.

Located as it will be, near the mouth of the Cumberland River, the Lower Cumberland project will be operated to produce substantial flood regulation on the lower Ohio and Mississippi Rivers, by reducing major flood crests. Provision of the connecting canal will enable flood waters from the Tennessee and Cumberland River Basins to be retained in both the Lower Cumberland Reservoir and Kentucky Reservoir, regardless of the geographic location of the flood-producing storm center.

The Lower Cumberland project is also an economical source of power generating capacity and is a logical step in the continuing program for comprehensive development of the Cumberland River. It will add 130,000 kilowatts of effective firm capacity to the existing power system of the Cumberland River, enhancing the dependability of the system's output and contributing materially to the available electric power supply of this region. In the production of hydroelectric power, the project will be somewhat self-regulating with utilization of stream flow as regulated by upstream reservoir units of the basin system. In addition, the interconnecting canal feature of this project makes possible the diversion of flow for power, thus affording further integrated operation of Lower Cumberland and Kentucky Reservoirs to maximize the benefits. The electrical energy output of the Lower Cumberland

plant, estimated as 600,000,000 kilowatt-hours average annually, can be used effectively through extensive existing transmission facilities of the Tennessee Valley Authority to serve the power needs over a large area.

The annual benefits from the three primary purposes are estimated as follows:

Navigation	$3,352,000
Flood Control	2,304,000
Power	3,836,000
	9,492,000

It is recognized that other benefits might be assigned to collateral uses such as recreation, water supply, and similar items, but the presently accepted policy is to forego these values since they are not as susceptible to evaluation tion as the primary factors.

A comparison of the annual benefits to the annual charges, the usual method of determining whether Federal expenditures are warranted, indicates a favorable ratio of 1.23 to 1.0.

Construction Schedule

Initial allotments for the project were obtained in the current 1956 fiscal year budget in the amount of $275,000. The approved budget for fiscal year 1957 as submitted by the President is $200,000, to be used for the continuation of preconstruction planning and preparation of plans and specifications. Under a realistic program based on experience in obtaining Congressional appropriations, approximately six years will be required for construction, which means that if first construction money is authorized in 1958, the project will not be fully operative until 1964. Efforts are now being made by local interests before Congressional Appropriations Committees to obtain initial construction funds in the 1957 budget which will permit a speed-up in final completion. The outcome of the requests, of course, will not be known until Congress passes the appropriation bill, anticipated just prior to the opening of the new fiscal year. The Corps of Engineers stands ready to prosecute the work to the extent of its ability within the limits of funds made available for the purpose.

TABLE 1 - PERTINENT DATA

GENERAL

PURPOSE

Flood control, navigation, and power.

AUTHORIZATION

River and Harbor Act of 1954 (Public Law 780, 83rd Congress, 2d Session)

LOCATION

The dam site is located at river mile 30.5 on the Cumberland River, in Lyon and Livingston Counties, Kentucky, about 8 miles west of Kuttawa, Kentucky, 24 miles northwest of Cadiz, Kentucky, and 22 miles southeast of Paducah, Kentucky.

RESERVOIR

OPERATING LEVELS:

Pool	Elevation, msl
Max. regulated for flood control, top of gates (96,000 acres)	375
Normal operation for navigation and power:	
Full pool (62,100 acres)	359
Minimum pool (50,600 acres)	354

STORAGE CAPACITIES (Flat pool assumption):

	Acre-feet (1)
Flood control (El. 375-359)	1,273,000
Power drawdown (El. 359-354)	282,000
Dead storage (below El. 354)	693,000
Total	2,248,000

LOCK AND DAM

DAM

Type: Combination rolled-earth fill and concrete gravity

Maximum height, foundation to top of dam, approx.	130

ELEVATIONS (MSL)

Top of dam (embankment)	385
Top of gates	375
Spillway crest	330
Base of dam (concrete section), approx.	265
Minimum tailwater	302

(1) Normal operation during non-flood season; flood control allowance to be increased to 1,555,000 acre-feet (El. 375-354) during season of major flood flows.

LOCK AND DAM (Continued)

LENGTHS IN FEET

Lock section, concrete	201
Spillway section, concrete	852
Power intake section, concrete	470
Right non-overflow section, concrete	188
Embankment sections, rolled-earth, approx.	6,274
Total, structures, approx.	7,985

SPILLWAY

Type	Concrete gravity, ogee
Total width between spray walls, feet	852
Net width between spray walls, feet	700
Crest gates, fixed-roller, vertical lift, number	20
Crest gates, height & width, feet	45 x 35

LOCK

Type	Concrete gravity; miter gates
Elevations (msl)	
Top of chamber and upper guide walls	382.0
Top of lower guide walls	350.0
Upper guard sill	339.0
Lower guard sill	288.0
Normal lift (feet)	57
Dimensions (feet)	
Clear width & length of lock chamber	110 x 800

CANAL

Uses	Navigation,diversion of flow for power and flood control.
Entrance at mile, Cumberland River	33
Entrance at mile, Tennessee River	25.2
Length of canal, miles	1.5
Bottom width, feet	600
Elevation of bottom, feet msl	330

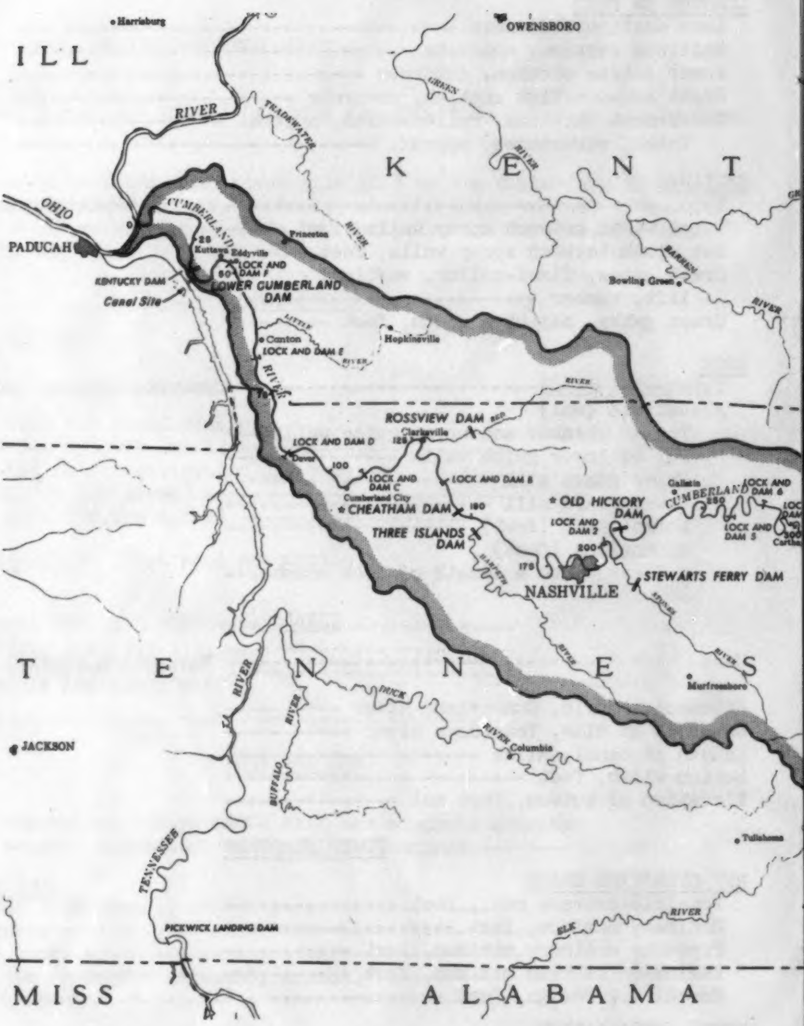
POWER FEATURES

NET OPERATING HEADS

Possible extreme max., feet	72
Ordinary maximum, feet	57
Probable ordinary minimum, feet	18
Estimated extreme minimum, feet	9
Normal for design, feet	44

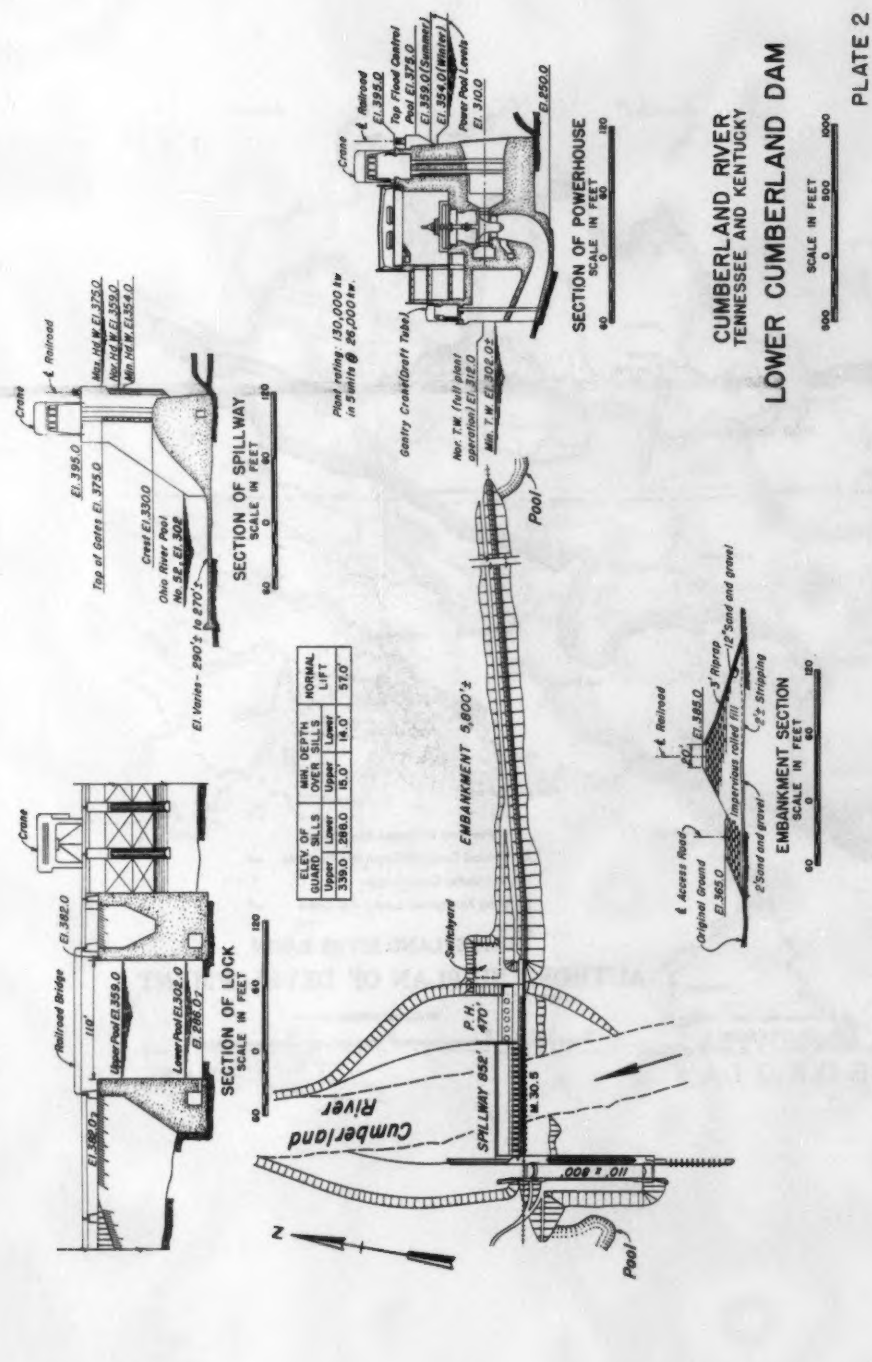
POWER INSTALLATION

Number of units	5
Unit capacity, kilowatts	26,000
Total capacity, kilowatts	130,000
Average annual energy output:	
From Cumb. River water, net kwh	480,000,000
From Tenn. River water thru canal, kwh	120,000,000
Total average annual, kwh	600,000,000


ILL
KENT
TENNES
MISS
ALABAMA
Harrisburg
OWENSBORO
PADUCAH
OHIO
RIVER
TRADEWATER
GREEN
RIVER
CUMBERLAND
Princeton
Kuttawa
Eddyville
LOCK AND DAM F
KENTUCKY DAM
Canal Site
LOWER CUMBERLAND DAM
Canton
LOCK AND DAM E
Hopkinsville
Bowling Green
BARREN
RIVER
ROSSVIEW DAM
Clarksville
LOCK AND DAM D
Dover
LOCK AND DAM C
Cumberland City
LOCK AND DAM B
CHEATHAM DAM
THREE ISLANDS DAM
OLD HICKORY DAM
Gallatin
LOCK AND DAM 2
LOCK AND DAM 6
LOCK AND DAM 5
NASHVILLE
STEWARTS FERRY DAM
HARPETH
STONES RIVER
Murfreesboro
DUCK
RIVER
Columbia
BUFFALO RIVER
JACKSON
TENNESSEE
PICKWICK LANDING DAM
ELK RIVER
Tullahoma

CUMBERLAND RIVER BASIN
AUTHORIZED PLAN OF DEVELOPMENT

PLATE 1



ELEV. OF GUARD SILLS		MIN. DEPTH OVER SILLS		NORMAL LIFT
Upper	Lower	Upper	Lower	
339.0	288.0	15.0'	14.0'	57.0'

LOWER CUMBERLAND PROJECT
CUMBERLAND RIVER, KENTUCKY
Artist's Perspective

PLATE 3

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

ARRANGEMENT OF GROINS ON A SANDY BEACH

Shositiro Nagai,[1] A.M. ASCE
(Proc. Paper 1063)

SYNOPSIS

It is an important and difficult problem to arrange groins effectively for protection against erosion by wave action on a sandy coast. This paper will present the relation of the groins' length, space, and orientation with respect to the shoreline, the direction of wave propagation, and the breaking point of the breakers. The relationship between wave steepness and sand transport, and some results of experiments concerning special types of groins are also presented.

INTRODUCTION

Groins, constructed perpendicular to, or at an inclination with, the shoreline, have been used as one of the most effective measures for coastal protection on a shore which has an abundant littoral drift coast. Notwithstanding a few laboratory experiments and field investigations with the groins,[2,3] it has, until to-day, been impossible to deal rationally with the length, direction with respect to wave climate, space and arrangement of the groins on sea shores. Therefore only practical experience has played an important role in this problem.

The writer has been studying since 1954 about the relationship between the length of groins, the space between them, the orientation with respect to wave propagation, the type and characteristics of waves, and the longshore currents due to wave breaking in the Field Hydraulic Laboratory of Osaka City University. Only the results of the experiments on groins will be described in this paper.

Note: Discussion open until February 1, 1957. Paper 1063 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 4, September, 1956.

1. Prof. of River and Harbor Eng., Faculty of Eng., Osaka City Univ., Osaka, Japan.
2. Proc. of the III Conference on Coastal Eng., Oct. 1952, pp. 137-163.
3. "Coastal Protection," by P. Bruun, The Dock & Harbour Auth., Nov. 1953, pp. 217-222.

Experimental Equipment and Procedure

Experimental Equipment

The experiments were performed in the Field Laboratory Basin shown in Fig. 1, approximately 40 m (131.23 ft.) by 15m (49.212 ft.) in plan and 0.30m (11.81 in.) at the maximum depth. The wave machine was of the flap type in which the period of the waves could be varied by changing the speed of the driving motor, and the height of the waves could be changed by adjusting the length of the crank arms connected to the wave flap. The period, length, and height of the waves were measured by four electric wave-height gauges which were constructed with two piano strings 2mm (.079 in.) in diameter and connected in series with an electro-magnetic oscillograph of 6 elements. The length of the waves was obtained by measuring the velocity of propagation between the two electrodes which were located a known distance apart. Two of the wave height gauges were set in the deep water basin to measure the wave height and wave length of the deep water waves, and two other gauges in the breaking point of the wave to measure the wave height of the breakers. The stretch of beach was about 30 m (98.42 ft.) in length, and built up to a layer of 3~5cm (7.62 - 12.70 inches) thickness of the ash-sands which were made in a melting-furnace and washed by water. The specific gravity of the ash-sand was nearly 2.2, and the mean diameter of the sand was about 0.6mm (.02 in.). Almost all the beaches were constructed with a slope of 1/20, and a few runs with a slope of 1/5, and the angles of the beaches to wave directions were 35^{o} to 90^{o}.

The period of the experimental waves was varied from 0.69 to 2.83 sec, the wave height in deep water from 1.2 to 5.9cm (0.47 to 2.32 in.), that in the breaking point from 3.9 to 8.4 cm (1.53 to 3.30 in.), the wave length from 0.83 to 4.00m (2.72 to 13.12 ft), and the steepness of the deep water waves from 0.003 to 0.045. The angles of the wave direction to the shoreline were 35^{o}, 48^{o}, 58^{o}, and 90^{o}, whereas the waves refracted at the angles of about 38^{o}, 53^{o}, and 63^{o} respectively near the breaking points on the 1/20 slope of the beach. The length of the model groins outside the shoreline was 30cm (11.81 in)), 50cm (19.68 in.), and 70cm (27.56 in.), and that inside the shoreline was all 20cm (7.87 in.). The top of the groins had a constant width of 2cm (0.79 in.) throughout the total length, and the same slope of the beaches, i.e. 1/20 slope, being set with a height of approximately 2cm (0.79 in.) above the still water level at the offshore end.

Experimental Results

1. Relationship between Wave Steepness and Sand Transport

It is a well-known fact that there are two types of sand transport; one is called littoral drift, which means a zigzag longshore transport of bedload debris on a beach slope directly due to a wave action, and the other a suspended drift, which means a transport of suspended material by a longshore current generated due to a breaking wave. Throughout the experiments, it was, as Mr. J. W. Johnson[4] and Mr. T. Saville[5] observed that the beach drift was

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4. "Sand Transport by Littoral Currents," by J. W. Johnson, Proc. of the Fifth Hydraulic Conf., June, 1952.
5. "Model Study of Sand Transport along a Infinitely Long, Straight Beach," by T. Saville, Trans. Amer. Geophys. Union, Vol. 31, No. 4, 1950.

predominant under the waves of small steepness which broke as a plunging breaker relatively near the shore, while the material transport in suspension was predominant under the steep waves which broke as a spilling breaker. When waves broke as a plunging type, they took up sands in water from the beds at the plunging points, and transported to the shoreline and deposited there, so that the slope of the beach steepened. Whereas the spilling breakers washed sands from the beach, and deposited it offshore or transported it in suspension along the shoreline, so that the beach profile flattened.

The change from a plunging breaker to a spilling appeared to occur at the steepness value of $\delta_o = 0.025$ in deep water waves. At the steepness values of $\delta_o \risingdotseq 0.01 \sim 0.02$ the sand transport towards the beach from the plunging points was the maximum, while the sand drifts decreased quickly for steepness ratios less than $\delta_o = 0.01$.

2. Length of Groins

The length of groins is one of the most important problems which should be decided in preliminary planning. This problem relates mainly to the slope and water depth of the beach, and the characteristics of the waves. It may be considered an appropriate and practical method to determine the length of groins by referring to the distance between the breaking points of plunging breakers and the shoreline. The results of the experiments conducted with this consideration are shown as follows:

a) As the offshore ends of groins approach the breaking points of plunging breakers, sand deposits increased on the down-drift sides, while scouring of the bed increased on the up-drift sides and near the offshore ends of the groins. The sand deposits on the down-drift sides and the scouring on the up-drift sides and offshore ends of the groins decreased when the groins were shortened by withdrawing their outer ends from the breaking points. From these facts it was considered that there should be a critical length of groin seaward out from the shoreline, which would be conducive to a maximum sand accumulation on the down-drift sides with a minimum scouring on the up-drift sides and near the outer ends.

b) Throughout the experiments it was proved that the length of such groins as mentioned above, was approximately 40% of the distance from the shoreline to the breaking points of the plunging breakers.

c) The length of groins shoreward from the shoreline, needs to be extended beyond the upper limit of the sand movement on the beach at the high tides.

From these experimental results it is considered that the length of groins should be determined with the observations of the breaking points of the predominant plunging breakers with the steepness of $\delta_o = 0.010 \sim 0.020$ on the shore and with the upper limits of the uprush of the waves at high tides.

3. Orientation of Groins

It is generally considered that the angle groins make with the shoreline must be changed according to the direction of the propagation of the waves to the shore.

1) Value of the Angle δ (Fig. 4). The value of the angle δ between the direction of the incident waves and that of the groins is one of the most important factors which influence the deposition and scour on the sides of the groins.

At the angle $\delta = 90^o$, a severe scouring took place on the offshore ends and up-drift sides of the groins, and a great wave pressure acted against the up-drift sides.

At the angles of $\delta < 90^o$, since the sands on the up-drift sides of the groins were transported towards the offshore, the beds on those sides were scoured.

Therefore it is considered that the angles of δ must be greater than 90^o, but when the values of δ were too large, the incident waves flowed towards the stem of the groins along the up-drift sides, scouring the shoreline at the stem. The experiments showed that the best value of the angle δ was

$$\delta \doteqdot 100^\circ \sim 110^\circ, \qquad (1)$$

and δ should not be greater than 120^o.

2) Value of the Angle α (Fig. 4). The angle α between the groins and the shoreline is determined from the angle θ of the incident waves to the shoreline and the condition (1). The values of α are shown in the Table-1.

But the greater values of α in Table-1 must be omitted, because a sand deposition on the down-drift sides of the groins decreases as a result of the diffraction and refraction of waves towards the down-drift sides. It is considered that the values of the angle α cannot be determined only from the angles of δ and θ, but must be decided also from another condition such as follows.

If it is assumed that when the area of the triangle Δ ABC (Fig. 4) is the maximum, the maximum sand deposition is produced on the down-drift sides of groins, it gives the condition;

$$F = \tfrac{1}{2} AB.BC \sin\beta = \tfrac{1}{2} L^2 \frac{\sin\beta . \sin(\theta+\beta)}{\sin\theta}$$

$$\frac{dF}{d\beta} = 0 \qquad (2)$$

$$\therefore \beta = \frac{\pi - \theta}{2}$$

in which F is the area of the triangle Δ ABC, and L is the length of $\overline{AB}$.

From the condition (2) the values of β and α shown in the Table-2 are obtained. According to the Table-2, the values of α which satisfy the condition (2) are $105^o \sim 120^o$ at the angles of $\theta = 30^o \sim 60^o$. It was verified that the condition (2) agreed well with the results of the experiments which were carried out at the values of the angle θ in the deep water, $\theta_o = 35^o$, 48^o, 58^o, and 90^o, and at the angles of $\theta_b \doteqdot 38^o$, 53^o, 63^o, and 90^o near the breaking point, and at the length of the groins L = 30cm (11.81 in.) and 50cm (19.68 in.). The experimental results were as follows;

a) At the Angles of $\theta_o \doteqdot 35^o$ and 55^o. Fig. 5 shows the scour and deposition on both sides of one groin. According to the Fig. 5, when $\alpha = 90^o$, waves rush towards the stem of the groin, eroding the stem and specially the bed at the offshore end. And since waves with comparatively great energy whirl into the down-drift side of the groin, little deposition occurs on this side. On the contrary, when $\alpha = 110^o$, waves move slowly towards the stem

of the groin, resulting in a small deposit, and since the whirling of waves with an adequately weak energy are caused on the down-drift side, there arises a good accumulation, resulting in an accretion to the shoreline on the side. When $\alpha \leqq 120^\circ$, there is only a small deposition on the down-drift side of the groin, since waves have a lesser tendency to assume a rotary motion.

Consequently when the angle θ_o was approximately 35° to 55°, the groins constructed to satisfy conditions (1) and (2), that is, the groins with the angle $\alpha = 105^\circ \sim 115^\circ$, as an average $\alpha_m \fallingdotseq 110^\circ$, caused a good deposition on the sides of the groins, and if the groins did not satisfy the condition (1) or (2), the deposition on the sides of the groins decreased.

b) At the Angles of $\theta \geqq 60^\circ$. When θ_o are approximately 60° to 90°, the groins can not satisfy simultaneously the two conditions of the angles $\delta = 100^\circ \sim 110^\circ$ and $\alpha < 120^\circ$, that is, when α takes values from 90° to 110°, δ yields values greater than 120°. Therefore, the orientation of the groins with respect to the shoreline which would give the least unfavorable effects for a sand deposition around the groins should be found at the angles of $\alpha = 90^\circ \sim 110^\circ$ by experiments.

From this condition it is seen that a sand accumulation near the shoreline due to groins would be more difficult at the angles $\theta \geqq 60^\circ$ than at $\theta_o < 60^\circ$. The fact was verified by experiments and also by cases on actual beaches.

More than thirty experiments were carried out to find the best orientation of the groins, the angles α of which were 90° and 110° to the shoreline, wave directions of $\theta_o = 60^\circ$ and 90°, and groins of 30cm (11.81 in.) and 50cm (19.68 in.) length. Sketches of the sand accumulation and erosion around the groins for $\theta = 90^\circ$, and the angles α of which are 90° and 110° to the shoreline, are shown in Fig. 6. This situation was nearly the same at $\theta = 60^\circ$. In general, when waves moved at the angles of $\theta \fallingdotseq 60^\circ \sim 90^\circ$ towards the shoreline, the erosion accurred at the stem of the groins, since the wave flowed to the stem along the sides of the groins, and a sand accumulation on the shoreline was slowed down owing to the fact that the return flow at the bottom prevented a deposition of the sands which had been transported from the breaking points towards the shoreline. The groins with the angle $\alpha = 90^\circ$ accumulated more sand on the shoreline than those with the angle $\alpha = 110^\circ$, as shown in Fig. 6, and this difference was distinguished at groups of more than two groins.

4. Space between Groins

Generally for a sand accumulation between groins, waves after breaking should penetrate in the area between the groins, carrying sand with the minimum energy required, and deposit the sand, with only water moving out in the return flow. If the space between groins is too great, the sand is removed from the area by waves with great energy, while contrary to this, if the space is too small, there is only a small accumulation of sand, since waves find it difficult to sufficiently penetrate the area between the groins to drop their load. Consequently it is considered that there should exist a optimum spacing in which the greatest amount of sand would be deposited. This spacing was investigated by the experiments.

1) At the angles of $\theta_o \fallingdotseq 35^\circ \sim 55^\circ$. The general aspect of the motion of water particles in the area between groins is shown in Fig. 7. The sand accumulation on the up-drift sides of the groins is similar to that of one groin

shown in Fig. 5, while the accumulation or the shoreline progress is very different from that on the down-drift sides in Fig. 5. It represents a distinguished effect of the second groin for the progress of the shoreline in the area between the groins. Fig. 7 also shows that the groins which are set at the angle $\alpha = 110^{\circ}$ to the shoreline are better than the groins set at $\alpha = 90^{\circ}$, while these are nearly equal in Fig. 5.

From the experimental observations the direction θ_s of the sand deposition to the shoreline in the area between groins was approximately

$$\theta_s \doteqdot 0.4\,\theta \tag{3}$$

If so, it is considered that the second groin is to be set at the point C shown in Fig. 8.

Then the distance D between the groins of AB and CD is

$$D = l\cos\beta + l\sin\beta \cdot \cot\theta_s \tag{4}$$

Since the angle $\alpha = 110^{\circ}$ was proved the best, β must be 70°. At the angle $\theta_o \doteqdot 35^{\circ} \sim 55^{\circ}$, the angle θ_s takes $\theta_s \doteqdot 0.4\,\theta \doteqdot 14^{\circ} \sim 22^{\circ}$, and therefore $\cot\theta_s \doteqdot 4.0 \sim 2.5$, the mean value $\cot\theta_s = 3.2$, the equation (4) yields

$$D \doteqdot l\cot\theta_s \doteqdot 3l \tag{5}$$

Throughout the experiments which were carried out concerning groins with the length, seaward of the shoreline, $l = 30$ (11.81 in.) 50 (19.68 in.) and 70cm (27.56 in.), and with the spaces, between the groins, $3l$, $4l$, and $5l$, the sand accumulation and the progress of the shoreline were proved to be maximum when the space was $3l$, and minimum when the space was $4l$ as shown in Fig. 9.

2) At the Angles of $\theta \doteqdot 60^{\circ} \sim 90^{\circ}$. When waves came towards the shore at the angles $\theta \doteqdot 60^{\circ} \sim 90^{\circ}$ to the shoreline, no longshore current occurred by a wave breaking, and a strong return flow towards the offshore appeared at the bottom, and hindered sand transport towards the area between groins. Therefore it was more difficult at the angles $\theta \doteqdot 60^{\circ} \sim 90^{\circ}$ than at $\theta < 60^{\circ}$ to give rise to sand accumulation and accretion on the shoreline, as mentioned above. From these experiments it was proved that the sand accumulation in the area between groins would be minimum when the space was $2l$, and maximum when the space was $4l$. The reason is that when the space is $4l$ the return flow is turned seaward from the shoreline, taking the cusp-like path as shown in Fig. 10, and only on the cusp-like path is sand removed from the bottom, to be accumulated to the seaward. Since the waves after breaking collide with the return flow near the offshore ends of the groins when the space is $2l$, the sand transported from the breaking point deposits near the offshore ends of the groins, while the return flow at the bottom is increased by the rise of the waterlevel near the shoreline. If the space was $5l$, there was a decrease of sand accumulation in the area between the groins, as the waves with great energy and the longshore current penetrated the area between the groins and removed sands from the shoreline. Fig. 10 shows the path of the motion of water-particles and the sand accumulation in the area between the groins which are set on the spaces of $D = 4l$, $3l$, and $2l$.

5. Special Types of Groins

When waves arrive at a sandy shore at nearly right angles to the shoreline, little or no sand accumulates in the area between groins as mentioned above. Specially on the coasts where storm waves with greater steepness than $\delta_o = 0.025$ may strike for considerably long hours at nearly right angles, it is known that the protective works should use such special types of groins as T-shaped or Z-shaped. The writer has carried out some experiments concerning the special types of groins. The results are as follows.

1) Submerged longitudinal structures. It was proved that if the top of the longitudinal structures provided at the offshore ends of groins was higher than MWL, that the longitudinal structures would be damaged on the offshore sides from a heavy attack of waves. It might therefore be better to build the top of the longitudinal structures submerged in water a little below MWL. If the longitudinal structures were connected at the outer ends of groins, there would be more sand accumulation in the area between the longitudinal works and the shoreline than there would be with T-shaped groins, because at T-shaped groins erosion occurred due to the concentration of the return flow from the shoreline to the mid-part where the longitudinal works were cut between the groins. The fact has also been proved by investigations on the Niigata coast[6] and other coasts.[7] According to the laboratory experiments and field investigations,[6] it may be desirable to construct the submerged longitudinal works as wide and permeable structures.

2) Revetment on the shoreline between groins. Some experiments, which were carried out concerning permeable and impermeable revetments against the erosion on the shoreline between groins when waves attacked the shore at nearly right angles, showed that when the tops of the revetments were higher than sea level the fore-sides of the revetments were eroded with wave attacks, while when the tops of the revetments were lower than sea level both sides of the revetments were eroded from the down-rush of water over the tops or through the structures. It means that the protection works against the erosion on the shoreline when waves attack the shore at nearly right angles should be an impermeable, high, and solid concrete sea-wall which waves may never overflow at high tides.

CONCLUSIONS

1) The littoral drift is predominant under the waves of small steepness which break generally as a plunging breaker near the shore, while the material transport in suspension is predominant under the steep waves which break generally as a spilling breaker on the offshore relatively far from the shore.

2) When waves break as a plunging type, they take up sand in the water from the ocean floor at the plunge point, and transport it to the shoreline and deposit it there, so that the slope of the beach steepens. Whereas the spilling breakers wash sand from the beach, and deposit it in the offshore or

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6. "Coastal Protection Works on the Niigata Coast," by S. Kuroda, The International Navigation Congress, Rome, 1953.
7. "Coast Protection," by R. Mimikin, The Dock & Harbour Auth., 1948-1949.

transport it in suspension along the shoreline, so that the beach profile flattens.

3) The change from a plunging breaker to a spilling appears to occur at the steepness value of $\delta_o \fallingdotseq 0.025$ in deep water waves. The sand transported towards the beach from the plunging points is maximum at the steepness values of $\delta_o \fallingdotseq 0.01 \sim 0.02$, while the sand drifts decrease quickly for steepness values less than $\delta_o \fallingdotseq 0.01$.

4) The length of groins seaward from the shoreline, where the maximum sand accumulation occurs on the up-drift sides and the minimum scouring is caused on the down-drift sides and near the offshore ends, is approximately 40% of the distance from the shoreline to the breaking points of the plunging breakers predominant on the shore. The length, shoreward from the shoreline, of groins needs to be extended beyond the upper limit of the sand movement on the beach at high tides.

5) When waves arrive at the shore at the angles of $\theta_o = 35^o$ to 55^o, it is most desirable for the sand accumulation in the area between groins that the angle α between the direction of groins and the shoreline is approximately 105^o to 115^o, as an average $\alpha_m \fallingdotseq 110^o$, and that the space between groins is 3ι, in which ι is the length of groins seaward from the shoreline.

6) When waves arrive at the shore at the angles of $\theta \fallingdotseq 60^o$ to 90^o, it is most desirable for sand accumulation that the angle α is 90^o and that the space between groins is 4ι.

7) The longitudinal structures provided at groins should be connected at the offshore ends of the groins, and are better constructed as wide and permeable structures. It is desirable for the top of the longitudinal structures to be a little lower than MWL on the shore.

TABLE 1

VALUES OF α

θ	δ	α	Remarks
30°	100° 110	110° 100	satisfy condition (2)
40	100 110	120 110	satisfy condition (2)
50	100 110	130 120	not satisfy condition (2)
60	100 110	140 130	not satisfy condition (2)

TABLE 2

VALUES OF β AND α

θ	β	α
30°	75°	105°
40	70	110
50	65	115
60	60	120

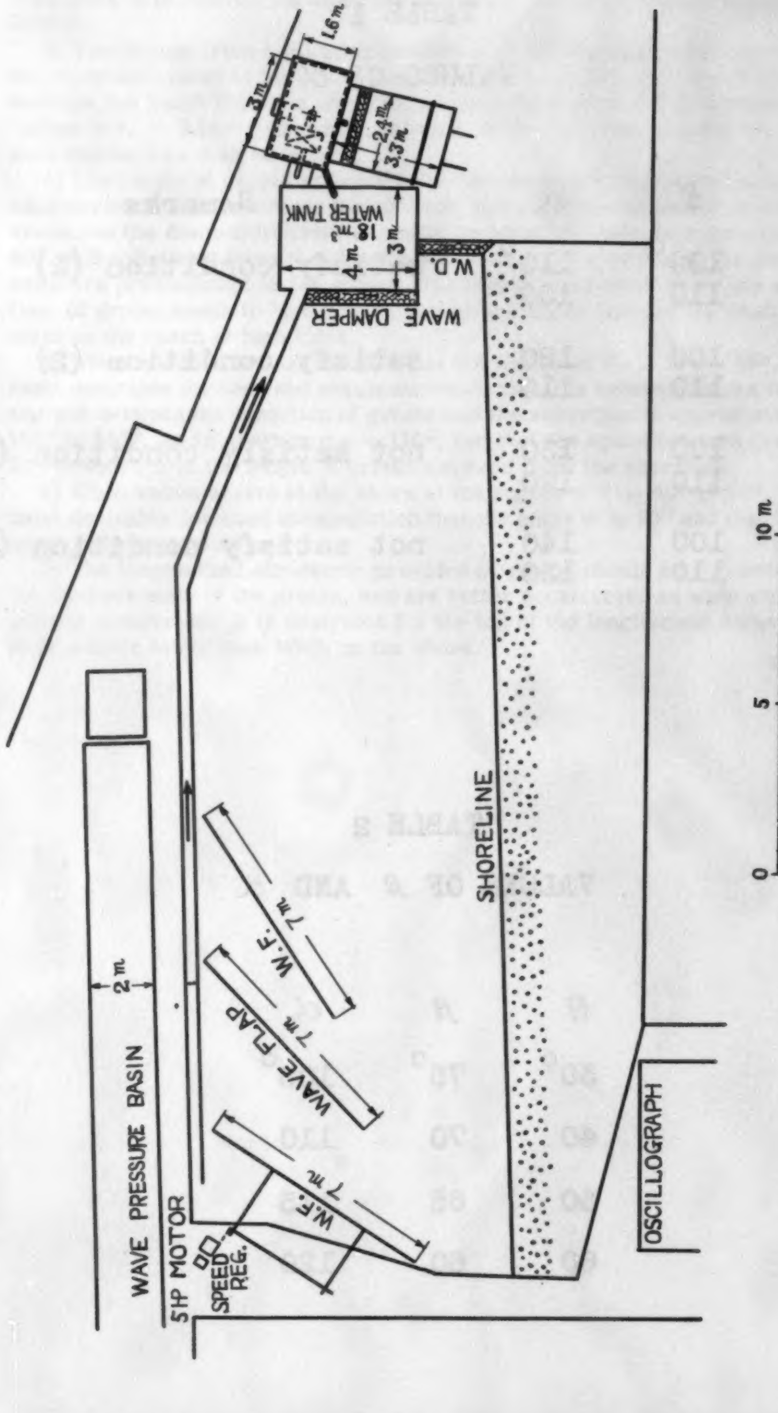

Fig. 1. Plan of the Field Labolatory Basin

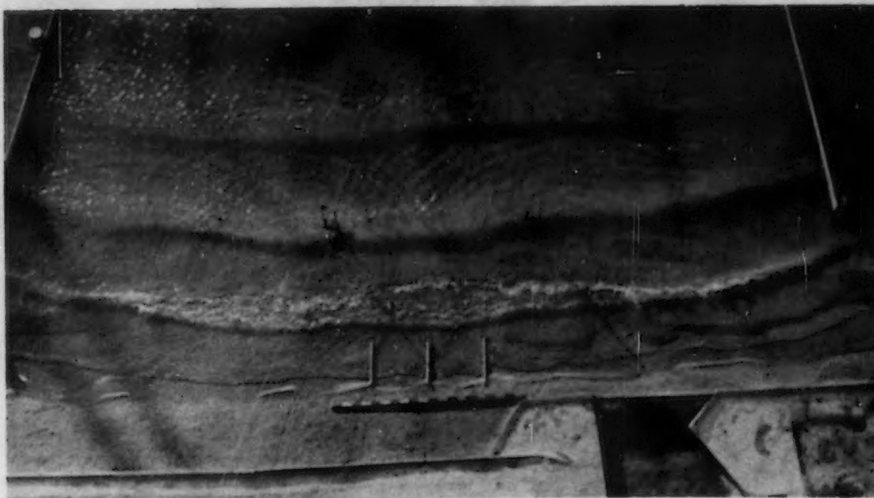

Fig. 2. Field Laboratory Basin and Model Groins

Fig. 3. Model Waves and Beach

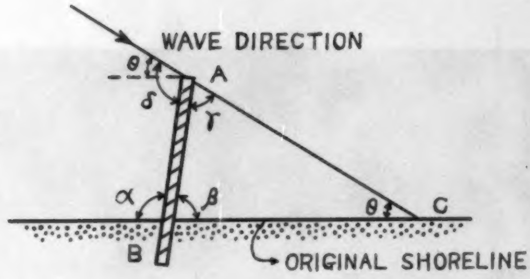

Fig. 4. Angles of Groins to the Shoreline.

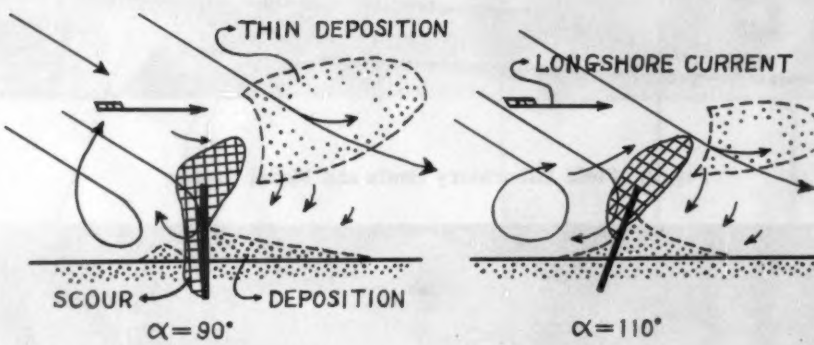

Fig. 5. Scour and Deposition on the sides of a Groin.

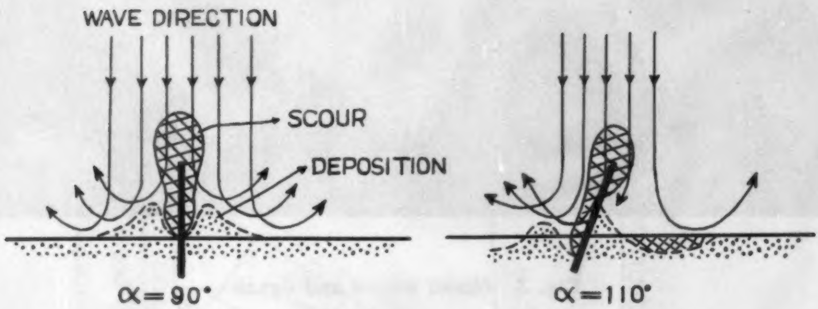

Fig. 6. Sand Accumulation and Erosion Around a Groin, at $\theta = 90^{0}$.

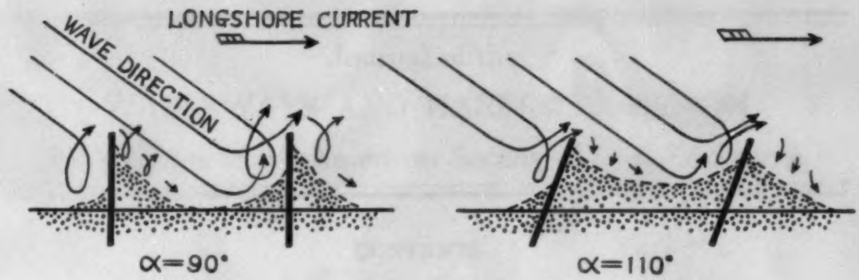

Fig. 7. Sand Accumulation in the Area Between Groins.

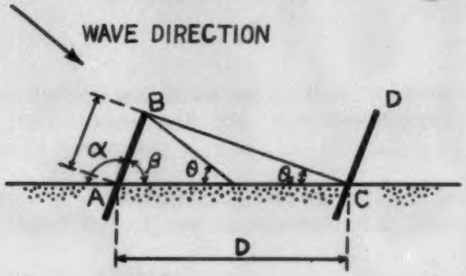

Fig. 8. Space Between Groins.

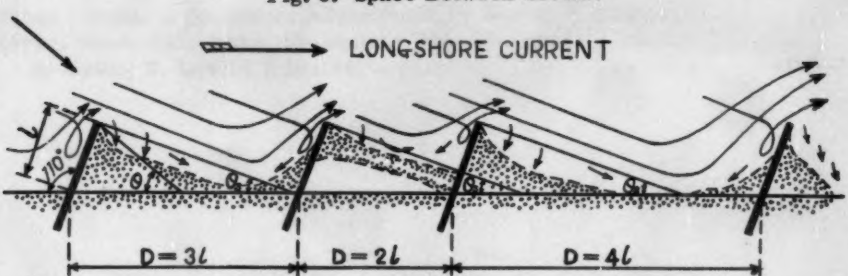

Fig. 9. Sand Accumulation in the Area Between Groins when D = $2l$, $3l$ and $4l$, at the Case of $\alpha = 110^{\circ}$.

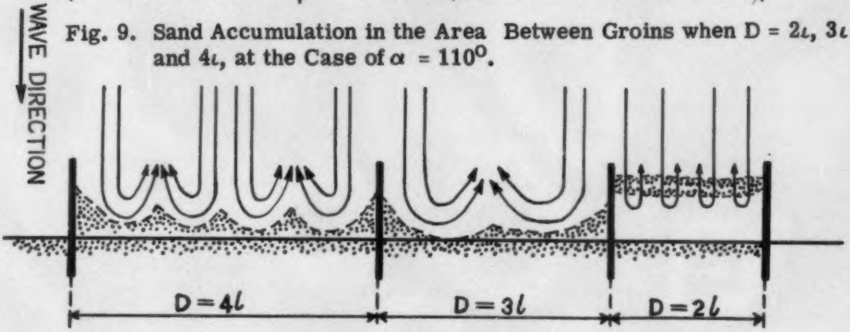

Fig. 10. Path of Water Particles in Motion and Sand Accumulation in the Area Between Groins, when $\theta \doteq 60^{\circ} \sim 90^{\circ}$ and $\alpha = 90^{\circ}$.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1068)

Page

The Design of Piers, Jetties, and Dolphins, by David A. Hopkins. (Proc. Paper 727. Prior discussion: 913. Discussion closed.)
by David A. Hopkins (Closure) 1068-3

Control of Arroyo Floods at Albuquerque, New Mexico, by Rufus H. Carter, Jr. (Proc. Paper 801. Prior discussion: 913. Discussion closed.)
by Rufus H. Carter, Jr. (Closure) 1068-5

Flood Control in the Middle Mississippi, by Walter F. Lawlor. (Proc. Paper 803. Prior discussion: 913. Discussion closed.)
by Walter F. Lawlor (Closure) 1068-7

Note: Paper 1068 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 4, September, 1956.

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1062)

Page

The Design of Piers, Jetties and Dolphins, by David A. Hopkins. (Proc. Paper 727. Prior discussion: 912. Discussion closed.)
by David A. Hopkins (closure) 1062-3

Control of Arroyo Floods at Albuquerque, New Mexico, by Dallas O. Carter, Jr. (Proc. Paper 801. Prior discussion: 915. Discussion closed.)
by Dallas O. Carter, Jr. (closure) 1062-5

Flood Control in the Middle Mississippi, by Walter P. Lawson. (Proc. Paper 818. Prior discussion: 915. Discussion closed.)
by Walter P. Lawson (closure) 1062-7

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Discussion of "THE DESIGN OF PIERS, JETTIES, AND DOLPHINS"

by David A. Hopkins
(Proc. Paper 727)

DAVID A. HOPKINS,[1] A.M. ASCE.—Mr. Levinton's correction of the tonnage terminology, which was erroneously described as "deadweight tonnage"—instead of "displacement tonnage"—is appreciated.

His discussion of the general considerations which affect spring fender design indicates an intelligent appreciation of fundamental principles, and raises a number of controversial points. It was certainly never intended that the paper should disparage the use of spring fenders. On rigid structures their value in minimizing impact loads on the structure or damage to the ship's hull is indisputable. Unfortunately, limitations of space preclude the possibility of a study of fender design in a discussion closure. For further study the reader is referred to the footnote references listed in the original paper dealing with the subject.

The following points, however, are selected for further comment:

1) The strength and flexibility of any particular structure are separate characteristics which are not inter-related in any way. Consider the analogy of an elastic spring, a spring may be designed for light or heavy loads, and also for small or large deflections. In the same way the rigidity or flexibility of a structure may vary, quite independently of its strength. Piers subjected to lateral impact loads commonly deflect 2 to 6 inches.
2) A sentence from the original paper—(referring to a preliminary design)—stated that "When . . . the comparative weakness of the ends of the pier was appreciated, it was decided that some stiffening of the ends was necessary." The use of the word "stiffening" in this case was possibly misleading, since it was used in the sense of spring stiffness (not structure stiffness). The "stiffening" referred to involved the substitution of heavier section H-piles at the ends of the pier as finally designed. The lateral load-carrying capacity of the ends of the pier was thus increased without any loss in flexibility.

   The problem of protecting the end of a pier is, of course, common to both flexible and rigid structures, since it is found in practice that the ends of piers are usually most vulnerable to accidental damage.
3) Deflections of pier structures up to 6 inches are common for piers under lateral impact loads. In the case of the Guayacan Bay pier, no difficulty was experienced in providing for 6 inches of movement when designing pipework and mechanical equipment, since the movement is gradual and the curvature of the structure extremely small. Present day design techniques call for a more careful evaluation of the effects of deflection than was customary in the past.

1. Design Engr., Mount Vernon, N. Y.

In conclusion, emphasis should be placed on the necessity for the absorption of lateral impact energy either by deflection of the structure or by spring fenders or by a combination of the two.

The examples described in the paper illustrate how structural deflection can be utilized advantageously to absorb this impact energy. The relative economy, however, of utilizing structural deflection as compared with spring fenders for the absorption of energy can only be evaluated by a study of particular cases. The two methods are available to design engineers, and no design study would be complete without assessing the extent to which either method is likely to be of value.

Discussion of
"CONTROL OF ARROYO FLOODS AT ALBUQUERQUE, NEW MEXICO"

by Rufus H. Carter, Jr.
(Proc. Paper 801)

RUFUS H. CARTER, JR.,[1] M. ASCE.—Mr. Allen's remarks concerning the design of the diversion channels for the control of arroyo floods at and in the vicinity of Albuquerque, New Mexico, are indeed timely and to the point.

To design these channels for supercritical flow under standard project flood conditions is especially desirable due to the resulting economies in construction; however, the sediment problem in the intercepted arroyos and in a diversion channel below any point of interception is not necessarily solved by inducing supercritical flow in the diversion channel under standard project flood conditions.

The usual concepts of suspended sediment and bed load fail to consider the accumulation of assorted debris, rocks, sand and soil grains that often ride a wave of initial flow in an arroyo. To transport any such accumulation of non-fluid solid matter through a transition curve and along a main channel will require a very careful analysis of the energy content in order that uniform or accumulating energy may insure the transportation of all material in the debris wave. A theoretical analysis together with supporting data from model studies should yield much valuable information toward the satisfactory design of these diversion channels and other channels for use under comparative conditions.

It is of interest that the Sandia Conservancy District has been organized under the laws of New Mexico for the purpose of meeting the obligation of local interests in the fulfillment of the Corps of Engineers' plan for control of arroyo floods. The District, through its consulting engineers, in cooperation with the City of Albuquerque, the County of Bernalillo, and various agencies of the State of New Mexico, has developed a plan of main drains or Primary Control Works that has been approved by the courts and is now an official plan of action. In the mesa area above the diversion channels these works will serve to collect storm flows from adjacent neighborhoods and deliver them safely to the Diversion Channels. In the valley areas these Primary Control Works will serve an identical purpose but will deliver the storm water to pumping stations for lift over levees into Rio Grande.

1. Civ. Engr., Corps of Engrs., U. S. Dept. of the Army, Albuquerque, New Mexico.

Discussion of

CONTROL OF ARROYO FLOODS AT ALBUQUERQUE, NEW MEXICO

by [illegible] L. Allen

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[1] [illegible] Corps of Engrs., U. S. Dept. of the Army, Albuquerque, New Mexico.

Discussion of
"FLOOD CONTROL IN THE MIDDLE MISSISSIPPI"

by Walter F. Lawlor
(Proc. Paper 803)

WALTER F. LAWLOR,[1] M. ASCE.—The intent of the writer in the statement queried by Mr. E. Kuiper was to point out primarily three general conditions which influence flood discharge stage relationships of the Mississippi River at St. Louis, Missouri, and which appear to explain why the discharge varied inversely with stage for approximately equal floods. The floods in question have the following characteristics.

	1943	1944	1947	1951
Stage at St. Louis	38.94	39.14	40.26	40.28
Flow	840,000	844,000	783,000	782,000
Primary source	Ill Riv & Lower Mo	Upper Miss	Mo Basin	Mo Basin
Time (Crest)	May 24	Apr 30	July 1, 2	July 21
Rate of rise (Bank full to crest)	7 days	8 days	24 days	24 days

From the above tabulation, the first inference drawn is that higher stages occur when a large percentage of the flow comes from the Missouri River; second, that the higher stages also occur during "summer" floods rather than "spring" floods and, third, that the higher stages also occur when the rate of rise is relatively slow. All three conditions are closely related, are generally common to the source of the flood, and all three are generally associated with a rise in the river bed. The practical use of the inferences would be in flood forecasting during critical stages of major floods at St. Louis.

In answer to the specific question of Mr. E. Kuiper, it was the intent of the writer to indicate that the higher stages were caused by a higher river bed and not by the alternate reasons suggested.

1. Chief, Eng. Div., St. Louis Dist., Corps of Engrs., U. S. Dept. of the Army, St. Louis, Mo.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

DESIGN CONSIDERATIONS FOR A NEW LOCK AT WILSON DAM

Robert A. Monroe,[1] M. ASCE, and George P. Palo,[2] A.M. ASCE
(Proc. Paper 1069)

SYNOPSIS

Navigation on the Tennessee River has increased to such extent that the present locks at Wilson Dam will soon be inadequate to handle the traffic. This paper discusses the major features of design of a new single-lift lock at Wilson Dam having a lock chamber 110 feet wide by 600 feet long and a maximum lift of 100 feet.

INTRODUCTION

The history of navigation on the Tennessee River dates back nearly 200 years. As early as 1785 settlers in Kentucky, Tennessee, and Alabama were using the river to ship produce to Mississippi ports. The main obstacle to the shallow-draft boats of that period was the rapids at Muscle Shoals in northern Alabama where the river dropped nearly 100 feet in a distance of a few miles.

Beginning in 1836 the construction of a series of locks around Muscle Shoals was attempted but the project resulted in complete failure, and shortly afterward a railroad was built around the Shoals which served for nearly 40 years.

In 1875 the U. S. Army began construction of a series of locks and canals which were completed in 1890. The section around Muscle Shoals had nine locks and 14.5 miles of canal and a project depth of 5 feet. Around 1900 the question of hydroelectric power development assumed importance, and over a period of nearly 20 years a series of bills providing for power development at Muscle Shoals were introduced in Congress, all of which failed to pass. Finally, in 1916 the National Defense Act provided for the construction of Wilson Dam which was completed with the present locks in 1925.

When the TVA took over in 1933, the three locks at Wilson Dam, a lock at

Note: Discussion open until February 1, 1957. Paper 1069 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 4, September, 1956.

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2. Head Civ. Engr., Tennessee Valley Authority, Knoxville, Tenn.

Widows Bar, and a lock at Hales Bar provided 6-foot navigation to Chattanooga. By 1945, with the construction of seven additional dams on the main river, a channel of 9-foot minimum navigable depth had been created extending 630 miles from the Ohio River to Knoxville.

Navigation on the Tennessee waterway is now around 20 times that of 15 years ago. The traffic through the Wilson lock totaled 2,210,000 tons in calendar year 1955. By 1958 it is estimated that the annual traffic will tax the capacity of the present Wilson locks to the utmost. The three small locks at Wilson Dam interpose a severe bottleneck to traffic. It requires six times as long for a tow to pass Wilson Dam as to pass Pickwick or Kentucky Dams.

A second lock is also urgently needed at Wilson Dam to handle traffic in case of a shutdown of the present locks for repairs. These locks are old and have mechanical and structural weaknesses which may require an outage any time.

In view of the increase in traffic and need for a standby lock, Congress in fiscal year 1955 and again in 1956 appropriated funds for design of a new lock. The design has progressed to a point where most of the main features are now determined. This paper deals with the major features of design as it has progressed up to April 1956.

Present Navigation Facilities

The present structures at Wilson are shown on photographs, figures 1 and 2. From figure 1 it will be noted that there is a double-lift lock at the right, or north end, of the Wilson Dam. The normal lift of the combined chambers is 90 feet. The upper chamber has a clear area of 60 by 300 feet and the lower chamber of 60 by 292 feet. From figure 2 it will be seen that immediately below the locks navigation enters the Florence Canal which follows the north shore of the river for approximately 2-1/2 miles. The canal is separated from the main channel of the Tennessee River (at this point at headwater of the Pickwick Landing Reservoir) by Patton Island. At the lower end of the canal and at the lower tip of Patton Island is lock and dam No. 1 which has a 10-foot lift and a chamber of 60 by 298 feet. The navigation requirement, therefore, to pass the Wilson Dam is three lockages with a total lift of 100 feet. It may be noted also from figures 1 and 2 that the navigation line through this area is tortuous. From the downstream end tows must pass under two bridges just before entering lock No. 1. At periods of high flow, lock No. 1 may be submerged and navigation interrupted for appreciable periods. When tows leave the upstream chamber of the lock in Wilson Dam, figure 2 shows that they are in direct line with a projection on the north bank of the Wilson Reservoir on which is built an important tower which is the anchor for the transmission line crossing of the reservoir. This condition, particularly at times of southerly winds, makes the exit from the upstream lock extremely difficult.

General Plan for New Development

The locks at the Pickwick Landing and Kentucky projects which are downstream from Wilson on the Tennessee River each have dimensions of 110 by 600 feet. Although some locks on the inland waterways are now being built to even larger dimensions, it seemed that continuation of this size was the

proper decision for the size of the new lock to be built at Wilson. Two general plans were then considered. Again with reference to figures 1 and 2 it was possible to:

1) Build a new lock at Wilson Dam with a lift of 100 feet; deepen the Florence Canal by 10 feet; eliminate lock and dam No. 1; build a third chamber in the existing lock at Wilson with a lift of 10 feet; or

2) Build a new lock at Wilson Dam with a lift of 90 feet; build a new lock adjacent to existing lock No. 1 with a lift of 10 feet.

The principal objections to scheme No. 2 were that this scheme would still require two lockages to pass the Wilson Dam and would require operating staffs at both locks. Furthermore, if existing lock No. 1 continued in even auxiliary service, fairly heavy maintenance items which have been deferred for a number of years in anticipation that this lock would be abandoned would have to be completed. It was therefore decided that the general plan of development would be in accordance with the procedure listed under scheme 1.

General Layout of Approved Scheme

Within the scope of the approved scheme two possibilities were considered:

1) Tributary stream control projects built since the completion of Wilson Dam make it possible to handle the maximum design flood through 50 spillway gates, whereas 58 were actually built in the original structure. The first design possibility as shown on figure 2 therefore located the center line of the new main lock in an area now occupied by the eight gates at the north end of the spillway. From the new lock traffic would enter the Florence Canal and follow the general alignment of this canal until at a short distance above lock and dam No. 1 it would enter a new alignment by which the canal bypassed the existing lock No. 1.

2) The main lock could be built in the same general position as scheme 1. Its lower downstream end would be swung a little to the south so that traffic leaving the lock would enter a completely separate canal cut generally through the center of Patton Island. This scheme would leave the existing lock system unchanged, and it could continue in auxiliary or emergency service.

Each of the above schemes had merit. Scheme 2 offered the straighter navigation line. The principal objection to this line, however, was that auxiliary navigation would still have to pass through existing lock No. 1. This lock is sufficiently remote from Wilson Dam to require that separate operating crews be maintained. As the comparative costs for schemes 1 and 2 were nearly equal, this factor became vital, and decision was made to carry out the project in general accordance with scheme 1.

Within scheme 1 there were four basic elements to the project:

A) The main lock and its equipment.
B) The 10-foot lift auxiliary chamber added to the downstream end of the existing lock.
C) The Florence Canal.
D) The revisions required to replace the existing highway bridge across the dam.

These features will be discussed separately but with some limitations. Major decisions are essentially complete on items C and D, and they will be discussed first. Many major design features of item A are still incomplete, and the discussion of that item must therefore be limited. Design of item B has received consideration to date only in connection with item A and will not be discussed separately.

Florence Canal

In the present navigation facilities the Florence Canal has a low water elevation 418 and provides a channel 11 feet deep and 300 feet wide. With the bypassing of lock No. 1 the low water in the Florence Canal will be elevation 408. To continue to provide an 11-foot navigation channel, it is therefore necessary to deepen the canal by 10 feet. Test pit and pumping experiments showed that the overburden in the island could be excavated to bedrock without leakage great enough to exceed capacity of normal construction pumping facilities. A number of years ago the TVA widened and deepened the Florence Canal by dredging. This work was accomplished while the canal remained open to traffic. It provided detailed information on the dredging problems and showed that at the upper end of the canal excavation was principally in rock whereas the lower end was principally in overburden. The excavation required, however, for deepening the canal will be a much larger job than was accomplished in the previous undertaking. The current project involves excavation of approximately 1,000,000 cubic yards of rock and 2,500,000 cubic yards of overburden. By a study of the problem it was found that a temporary canal could be excavated through Patton Island, bypassing the portion of the existing Florence Canal which would continue in use after the new lock project is completed. By means of this temporary canal and modest dikes it is possible to continue navigation but to unwater the existing canal so that the major part of the excavation, particularly of the rock, can be made in the dry. At the lower end of the canal the alignment has been carefully chosen so as to provide the best possible sailing line past the swing span in the railroad bridge. Most of the excavation will be piled on Patton Island. This excavation will raise a dike already built on this island which protects the canal from high water flow in the Tennessee River. At times of high flow, eddies develop at the lower end of the island thus complicating the difficulties of tows as they attempt to pass the railroad bridge. The design program plans a model in the hydraulic laboratory to study this condition to determine whether excavation from the canal can be deposited in a way which will improve this eddy condition. Figure 3 shows the route of the reconstructed canal.

Highway Bridge Across Dam

The existing Wilson Dam has a 20-foot-wide highway bridge on its deck. This bridge crosses the existing lock on a bascule span. Traffic measurements indicate approximately 1800 vehicles each day, much of which concentrates at times of shift change in the work not only on the TVA reservation but in the adjoining industries. It is probable that this traffic will increase, but at the same time it does not seem that the crossing of the Wilson Dam can ever be converted reasonably into a high standard bridge. Therefore, the

major traffic must continue to cross the river on the bridge just downstream from Patton Island or on a second bridge which is already being discussed for that area. It was therefore decided that the crossing of the new lock would continue to provide a 20-foot roadway and a continuation of the sidewalk which now exists downstream from the roadway on the present dam.

Four bridge possibilities were considered:

1) A high-level bridge.
2) A low-level bridge on the present highway alignment crossing the new lock on a bascule span.
3) The same as (2) but with a lift span.
4) A low-level bridge crossing the dam on the existing roadway and with two right-angle turns just south of the new lock so as to provide an alignment which would cross the locks at grade but provide satisfactory navigation clearance above the tailwater below the locks.

There is a natural desire to avoid the interruption to traffic inherent in movable bridges. The sharp bends in the alignment of scheme 4 were considered at first no more serious than the tortuous approach to the dam at the powerhouse end. However, as design studies continued it was found possible to improve the approach at the powerhouse, and under those circumstances it was considered undesirable to introduce poor alignment in the crossing of the lock. Careful cost estimates were made for all four schemes, and annual charges against each were determined. A modest allowance was included for the cost of interruption of traffic in the two schemes using movable bridges. The lowest annual cost was obtained for scheme 4, but, as already stated, the alignment of this scheme was undesirable, and as the annual cost for scheme 1 was only slightly higher this scheme was chosen.

The adopted high-level bridge consists of a series of beam spans and girder spans. Grades were limited to 5 percent. The start of the ramp from the existing spillway deck is constructed with concrete parapets generally similar to those now existing on the dam. After the roadway rises several feet above the level of the spillway deck, the railing changes to a modern open-type steel handrail. The alignment of the bridge after it approaches the locks is slightly downstream from the existing highway so that the new bridge passes downstream from the existing lock operation building and strikes its north abutment at a favorable high place on the north shore of the river.

Main Lock

Three principal factors introduce a number of unusual features into the design of the main lock and its equipment. These factors are:

1) The single lift of 100 feet. It appears probable that upon its completion the Wilson lock will be the highest single lift lock in existence. The Corps of Engineers, however, is now studying the Ice Harbor lock on the Snake River which will have a lift of 102 feet but a width of only about 85 feet.
2) Construction of the lock in an existing dam.
3) The existence of the long canal downstream from the lock. This results in a situation in which the level of the water in the canal varies up to a maximum of about 15 feet below the level of the water in the river along the river wall of the lock. This condition, as will be noted below, affects considerably the design of the lock-emptying system.

Lock Walls

Figure 4 shows plan and typical sections of the main lock walls and of the guide and guard walls. Foundation rock is excellent but at an elevation slightly higher than might be considered most favorable, and, therefore, a considerable amount of rock excavation will be necessary before concreting can begin. The main lock walls are conventional. The downstream guard and guide walls are also rather conventional sections. It may be noted that the downstream end of the downstream guard wall coincides with an existing retaining wall which outlines the present channel and may be noted in the foreground of the photograph, figure 1. During the various construction phases, the downstream guard wall will be required to function as a cofferdam with water nearly to its crest on the river side. This creates the largest load which this wall may have to carry. The upper guard wall is upstream from the existing dam and, therefore, is located in the reservoir which is approximately 110 feet deep. A floating structure will be required and the guard "wall" will be actually a guard "boom." A concrete floating guard boom was used at the Kentucky lock and has functioned very effectively. At Kentucky, concrete was used in preference to structural steel because it was built during World War II and structural steel was not available. It is not as yet decided what materials will be used for the Wilson guard boom. It would appear that a design of steel will result in a less expensive structure. However, such a structure would be lighter and, therefore, might not have the same stability during heavy storms as does the more massive concrete boom at Kentucky. It is possible also that continual striking of the boom by barges entering the lock may, despite protection of timber fenders, create more eventual damage in a steel boom than in a concrete boom. Finally, a steel boom would undoubtedly have to be dry-docked occasionally for painting and maintenance. This could be done effectively in one of the chambers of the existing lock. All of these factors will be given further consideration in the design office and in the hydraulic laboratory before final decision on the materials of which the boom will be built is made.

Filling System

The construction of the lock in an existing dam creates major problems in a design of the filling system. To fill this lock within a reasonable time, a maximum inflow of about 20,000 cfs will be required. The conventional procedure by which large submerged inlet ports are built into the face of the lock wall upstream from the upper gate sill cannot be adopted for this lock. It is possible to provide culverts which extend directly through the existing spillway section to an inlet area on the upstream face of the existing spillway dam. Such a system might have the upstream end of the culvert approximately 15 feet wide and 23 feet high resulting in average velocity of about 29 feet per second if 10,000 cfs is drawn into the culvert in each wall. The bottom elevation for the bellmouth at the entrance to this culvert would be at approximately elevation 440 if negative pressures at the culvert entrance are to be avoided. This bottom elevation is 68 feet below the surface of the Wilson Reservation and obviously involves special cofferdam problems for construction. The concept of cofferdamming would be a bulkhead type of box which could be sunk and sealed against the upstream face of the dam and then unwatered to provide minimum working areas. The design would also require construction of trashracks of areas suitable for reasonable entrance

velocities. It has not yet been determined whether such racks could be built economically within a cofferdam or whether some type of prefabricated system would be built and then placed against the upstream face of the dam by the use of divers.

The cofferdam complications involved in the above scheme made it advisable to consider the possibility of schemes which would permit construction of the new lock without use of cofferdams. As noted earlier, the lock will replace the portion of the spillway dam now occupied by spillway gates 1 to 8, inclusive. The lock structure actually occupies space only of gates 3 through 6. It was, therefore, possible to consider filling the lock by utilizing all or part of existing spillway gates 1 and 2 for the land wall culvert and 7 and 8 for the river wall culvert. A scheme was developed for filling the river wall culvert through only gate 8. The discharge capacity of this single gate is approximately 10,000 cfs, or the amount needed for proper filling of the lock. It was recognized that filling from the reservoir surface through a culvert with several bends involves serious complication of air entrainment. However, if the system could be made to work, it had two principal merits:

1) It avoided the need for cofferdams during construction.
2) It avoided the need for a conventional-type reversed taintor gate in the filling culvert. Although this type of gate has been used effectively in many locks, there is evidence that the head can be increased to a point of serious cavitation on such gates unless considerable aeration is provided. If aeration requirements are too great, then the mixture of air and water which is introduced into the lock chamber results in highly undesirable boils in the lock during the filling operation.

To check the proposed design by model testing it was necessary that the model scale be large enough to obtain some concept of the air entrainment problem. A model scale of 1:16, the largest practical in the laboratory, was adopted. As this paper is being prepared, initial model testing has been completed. The air entrainment in the first test was so serious that it appears that this scheme must be abandoned. It appears necessary, therefore, to introduce into the filling culvert the conventional type of reverse taintor gate. Careful model studies will be made so that the structural details of this gate and the means for its aeration may be carefully checked. After the first model failed to function satisfactorily, careful consideration was again given to providing a culvert through the dam as has already been described. In addition, a scheme was developed by which the river culvert would be filled by discharge over both spillway gates 7 and 8 and the land wall culvert through spillway gates 1 and 2. The culvert through the dam appeared to offer both the best and cheapest solution to the filling problem, and the hydraulic laboratory is now proceeding to develop the model for test on that basis.

Final discharge into the lock will be by means of a bottom lateral system. Present designs call for a total of ten laterals, five connected to each main wall culvert. Each lateral has twelve ports. Culverts are 15 feet square. Entrances to laterals are 9 feet high and 6 feet wide, and each of the twelve ports is 3-1/2 feet high and 1-1/2 feet wide. A single lateral has been tested in the laboratory to develop these dimensions and to shape the ports so as to provide as uniform distribution of discharge as possible.

Emptying System

As already noted the fact that the main river is higher than the canal immediately downstream from the lock prevents the complete emptying of the lock into the river. To empty the lock at a rate of 20,000 cfs into the canal was always considered to develop a serious condition of turbulence and eddies near the lock entrance and swells in the canal. Accordingly, it seemed possible to consider only emptying schemes which would either:

1) Empty the land wall culvert into the canal and the river wall culvert into the river, or
2) Empty both land and river wall culverts into the river and provide supplemental emptying into the canal to permit removal of the last several feet of water in the lock.

Even the discharge of only the land wall culvert into the canal appeared to create considerable problem of turbulence and eddies. It was found impossible to make further check in the hydraulic laboratory on the actual conditions of discharge in this scheme until too late in the design schedule; therefore, the alternates of 2 were considered essential for final design. Cost estimates showed that the supplementary emptying system could be built for approximately the same cost as scheme 1 because of heavy excavation which would have been required at the outlet end of the land wall culvert in that scheme. It was, therefore, decided that both main lock culverts would discharge into the river. Supplementary emptying will be through culverts around the lower gate of the lock.

Main Downstream Gate

Assuming a conventional type of miter gate, the great lift in the lock requires a downstream gate 117 feet high. There are two conventional types of gates—one utilizing a straight girder, the other an arched girder. The latter permits a gate of lighter weight, but the fabrication is more complicated and was considered to result in a more expensive gate. However, because of the high lift, the top 37 feet of the gate actually could be replaced by a fixed structure and still provide satisfactory navigation clearances. It was therefore possible to consider the construction of a fixed wall above elevation 475 to replace the upper part of the conventional gate. To the best of our knowledge, the only modern lock on which such construction has been adopted is the Mondragon lock on the Rhone River in France. This lock, however, is only about 39 feet wide. In making studies of a fixed wall either of steel or concrete, modest economy seemed possible, but extremely difficult problems seemed to develop in sealing the upper part of the gate to the lower part of the wall and in locating the wall in such position relative to the gate that suitable access could be obtained to the gudgeon pin and anchorage bars. After study for an appreciable time it was decided that no suitable solution to these problems could be effected which would justify the use of a fixed wall to replace a part of a typical conventional miter gate. However, if a lift gate were used a fixed wall appeared possible. The lift gate with the machinery necessary to lift its weight 80 feet, required to open the gate, was a tremendous structure. However, a lift gate might result in two economies in addition to replacement of a miter gate. First, it would be possible to effect the final emptying of the lock simply by cracking the lift gate and thus save the cost of bypass culverts which were considered necessary with a miter gate.

Incidentally, either sluice gate or butterfly gate openings through a miter gate were considered but did not seem as desirable as bypass culverts. Second, since a lift gate does not require space in which to swing open, the total length of the main lock walls could be reduced by about 30 feet as compared to the length required with a miter gate. Despite these benefits the lift gate and fixed wall were found to cost $250,000 more than a conventional miter gate. A miter gate was therefore adopted.

Main Upstream Gate

For the upstream gate of the lock the conventional miter type as well as the lift and taintor types was considered. Both of the two latter types are lowered to open the gate. Miter and lift gates have been used on all widths of locks and the taintor gate has been used, though apparently not to date, on locks 110 feet in width. The choice among these three types of gates was a narrow one. The lift-type gate was chosen principally because it appeared that this gate and its operating machinery would be beneficial to the over-all arrangement around the upper gate sill. The lift gate permitted ample room for emergency closures and for the possible use of a culvert valve in the lock wall adjacent to the upper gate sill. Actually it does not appear that final design will locate the culvert valve at this point. The lift gate also permitted the placing of all these features at a point relatively clear from interference with the bridge crossing above the lock.

The lift gate as designed consists of conventional welded girders with wheels operating on a track in recesses in lock walls. There is an overflow crest on the gate so that it may be used to supplement filling during the final minutes of the lock-filling operation. The two lower girders of the gate are connected with plates so as to provide a buoyancy chamber reducing the net weight of the gate when it is submerged. The gate-operating equipment in the two lock walls will have electrical inter-connection to ensure that the gate is raised uniformly at both ends.

Emergency Closure

For repair of the main lock gates emergency closures are required. The closure at the upstream end is about 21 feet high and at the downstream end 26 feet high. Poiree or needle dams were considered undesirable for these closures. Steel or aluminum truss stoplog sections for the full width of the lock were seriously considered, but the Corps of Engineers who must maintain the lock pointed out that their available derrick boats did not have capacity for handling the weight of such stoplogs. At their suggestion a scheme was developed by which a central support pier of steel beams will, during normal operation, lie flat in a recess in the gate sills. When needed this frame can be lifted and locked into position. Then stoplogs consisting of steel trusses of length equal to one-half the width of the lock can be placed to effect the desired closure.

When the Kentucky lock was constructed, consideration was given to the fabrication of a floating caisson which would be used for emergency closure. Such a caisson was never constructed, but the lock was built with necessary recesses in the walls for the seating of the caisson. A single caisson might be used not only for the Kentucky lock but for other new 110-foot-wide locks on the Tennessee River and for new locks built by the Corps of Engineers on the Cumberland River. The Corps has provided for such a caisson at the

Cheatham lock nearing completion on that river. The same provisions for a caisson will be made at the Wilson lock. Provision can be made at little cost and if the caisson is fabricated at a future date, it can replace in part the need for stoplogs at the Wilson lock or at possible future locks on the Tennessee.

CONCLUSION

The President's budget for fiscal year 1957 includes funds for beginning construction of the new lock. If Congress approves such an appropriation, under the proposed construction program the new lock and canal should be opened to operation in the fall of 1959. Construction of a third chamber on the existing lock will require 1-1/2 years longer, making a total required construction period of 4-1/2 years. The total cost of the completed work is estimated to be 35 million dollars. An artist's conception of the completed lock superposed on a photograph is shown on figure 5.

The Nashville District of the U. S. Army Engineers handles the operation and maintenance of all locks on the Tennessee River. The general plans for the new lock at Wilson Dam as affecting operation and maintenance have been closely coordinated with the Nashville District. Much valuable information on recent new high-lift locks has been obtained from other district offices of the U. S. Army Engineers. Planning studies for the new lock were made by the Water Control Planning of TVA, directed by James S. Bowman, M. ASCE and Reed Elliott, M. ASCE. Design is being handled by the Division of Design of TVA, and construction will be handled by the TVA Division of Construction under the direction of George K. Leonard, M. ASCE. All planning, design, and construction work is under the supervision of C. E. Blee, M. ASCE, Chief Engineer of TVA.

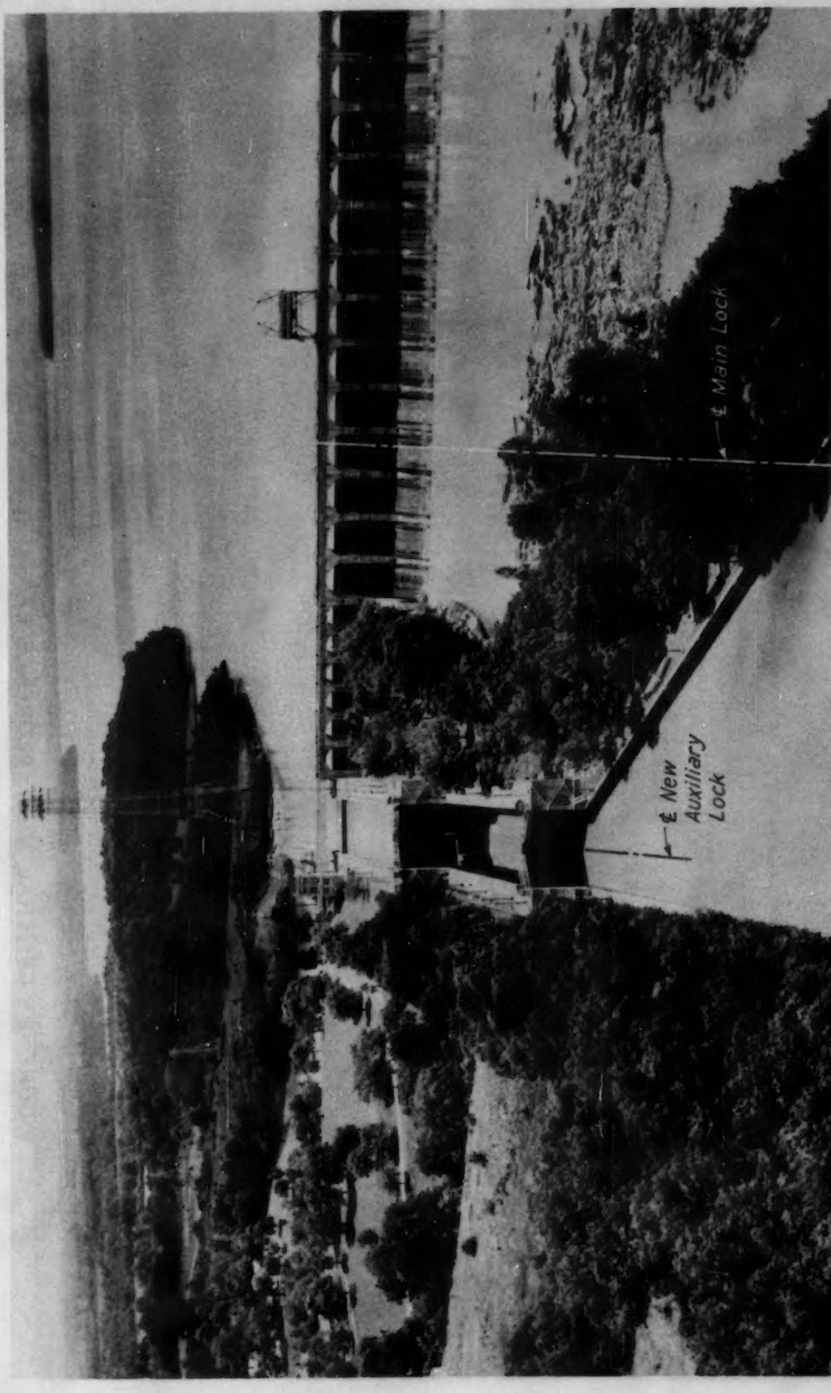

FIGURE 1 - VIEW OF WILSON DAM SHOWING LOCATION OF PROPOSED LOCKS

FIGURE 2 - GENERAL VIEW - FLORENCE CANAL AND WILSON DAM

FIGURE 3-ARTIST VIEW NEW FLORENCE CANAL

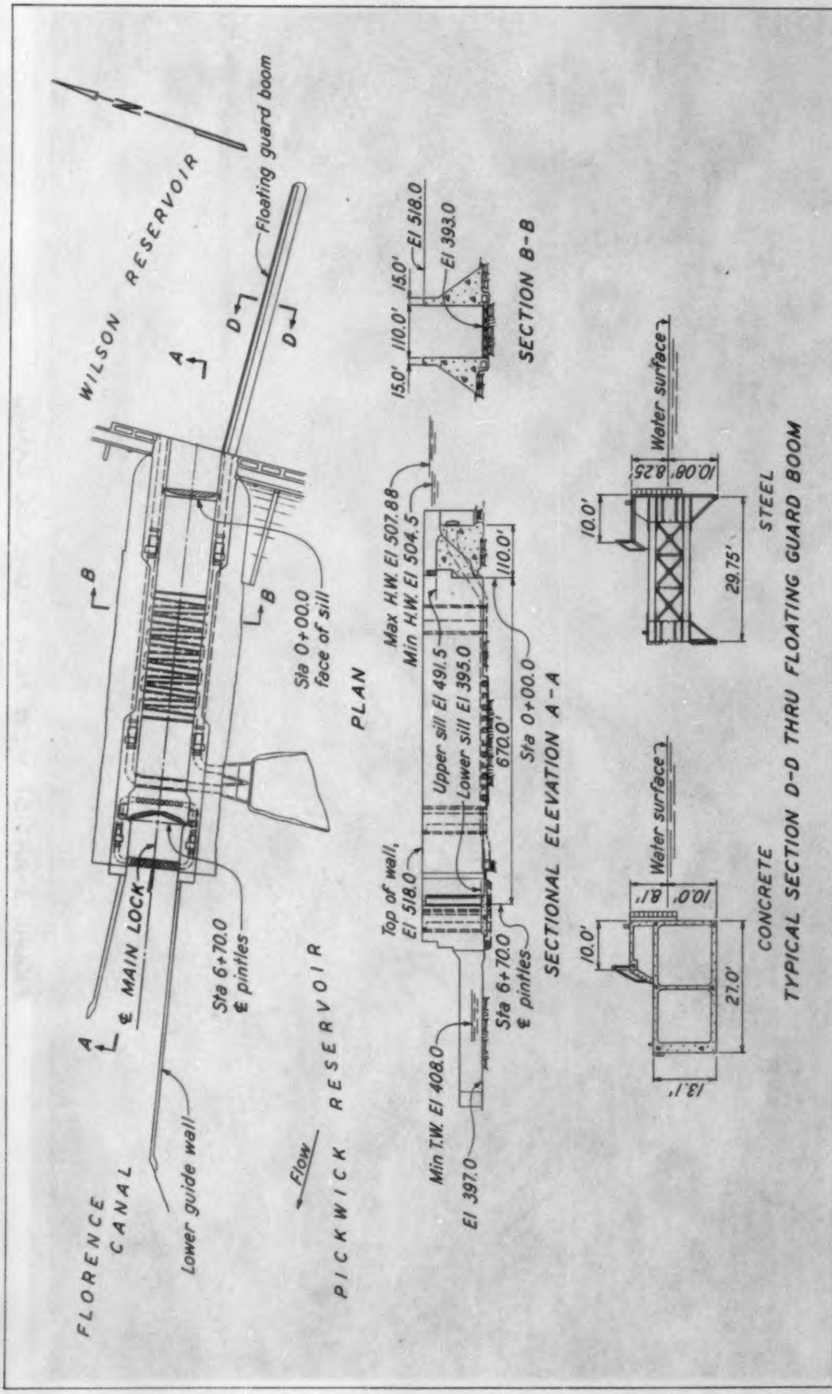

FIGURE 4 - GENERAL PLAN NEW WILSON LOCK

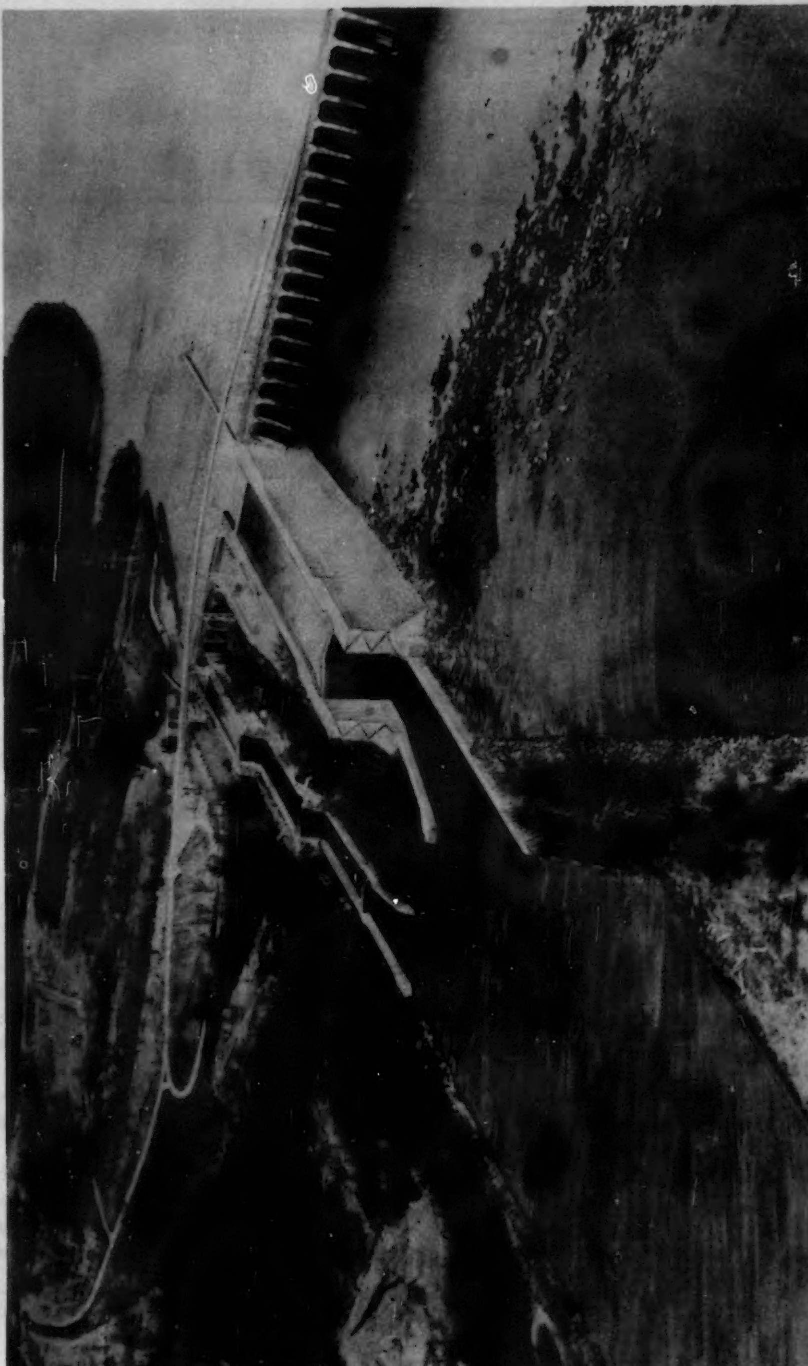

FIGURE 5-ARTIST VIEW NEW LOCKS AT WILSON DAM

DIVISION ACTIVITIES
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

NEWSLETTER

September, 1956

MARCH MEETING OF EXECUTIVE COMMITTEE

The meeting of the Division Executive Committee was held on March 16, 1956, at ASCE headquarters, New York. Those present were:

Members of the Executive Committee

Charles M. Wellons, Chairman
Lewis C. Coxe, Vice Chairman
Roger H. Gilman, Secretary
Carey H. Brown, Contact Member from the Board of Direction

Chairmen of Administrative and Technical Committees

Herbert D. Vogel, Chairman, Committee on Research
Norman R. Moore, Chairman, Committee on Cooperation with Local Sections
Frank W. Herring, Chairman, Committee on Ports and Harbors
Charles F. MacNish, Chairman, Committee on Design, Construction, and Operation of Navigation and Flood Control Locks and Dams
Austin E. Brant, Jr., Newsletter Editor

At the outset, Chairman Wellons welcomed General Vogel and Mr. Moore as the new Chairmen of their respective Committees. Mr. Carey H. Brown has recently been appointed the Contact Member of the Board of Direction under the new ASCE procedure whereby each of the technical divisions will have such a Contact Member. It is particularly appropriate to have Mr. Brown serve with our Committee in view of his 20 years of association with the U. S. Army Corps of Engineers as well as his service as Chairman of the Task Group on navigation matters for the Hoover Commission.

Chairman Wellons reviewed the activities and the important developments with relation to the Division since our October meeting. Of unusual importance was the approval by the Board of Direction at its February 1956 meeting of both recommendations adopted by the Executive Committee of the Division last October. First, the name of the Division has been changed officially

Note: No. 1956-21 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, WW 4, September, 1956.

to Waterways and Harbors Division to reflect more accurately the scope and interest of our activities. Second, the establishment of a Committee on Research was also authorized and the Executive Committee has approved the designation of General Vogel as Chairman of that new technical committee. One of the first tangible results of the expansion of the title of the Division has been the establishment of a local Waterways and Harbors Committee within the Maryland Section of the Society because of the special interest engendered by the Port of Baltimore's current plans for port development.

Chairman Wellons commented on the activities of several of the technical and administrative committees during the past six months, particularly the Committee on Design, Construction, and Operation of Navigation and Flood Control Locks and Dams, which under the new Chairmanship of Mr. MacNish held a meeting in Buffalo last December where a program of activities, new papers, participation in convention sessions, etc., was drawn up.

The responsibilities and possible program of the Committee on Cooperation with Local Sections were reviewed. Approval was given to including on Mr. Moore's Committee, Col. James Holcombe, Assistant District Engineer of the Corps of Engineers in Buffalo, whose membership will be particularly helpful in connection with planning the Buffalo meeting of the Society in June 1957. There was agreement that wherever possible the Committee should stimulate interest in the local sections to form special Waterways and Harbors Committees which would help to develop programs and promote a greater Division membership within the local section areas. Such local Division activities as well as the individual contact members serving on the Committee on Cooperation with Local Sections will also assist in obtaining personal items which can be included in the Division Newsletter.

Although he was unable to attend the meeting, Mr. Evan Vaughan reported on the activities of his Publications Committee. A total of eight papers were carried over since September 1955 of which four have been cleared for publication; three are in the hands of authors for revision and one, which was prepared for a joint session, has not yet been received from the Hydraulics Division. Since September, 20 new papers have been received of which 17 have been cleared for publication (including the five delivered at the Bridge Clearance symposium in October 1955) and three are in the process of review. Mr. Vaughan also made special note that there has been a substantial improvement in the quality of the papers which his Committee has been reviewing.

Mr. Gilman also reported on the discussions held last November with the American Association of Port Authorities which indicated their approval of closer liaison on matters of mutual interest. Such liaison will be carried on mainly by the Committee on Ports and Harbors.

PITTSBURGH CONVENTION

The Committee on Session Programs of the Waterways and Harbors Division has scheduled four sessions for the Pittsburgh Convention. The morning session of October 15 will be devoted to navigation and flood control on the Ohio River and tributaries. The afternoon session on the 15th has been arranged by the Division Committee on Ports and Harbors. The morning and afternoon sessions of October 16th are jointly sponsored by the Waterways and Harbors and Hydraulics Divisions. These sessions consist of six papers on coastal engineering subjects.

The Waterways and Harbors Division, the Structural Division, and the Hydraulics Division will jointly sponsor a luncheon on Tuesday, October 16th. The luncheon will be addressed by Lt. Gen. Sturgis, Chief of Engineers, on the new construction underway and proposed for the Ohio River system.

The program for the Waterways and Harbors Division sessions is as follows:

Monday, October 15, 1956 - Morning Session

Presiding: Charles M. Wellons, Chairman, Executive Committee, Waterways and Harbors Division

"Modern Navigation Facilities for the Ohio River and Certain Tributaries"

James W. Bruce, Chief of Planning and Reports Branch, Ohio River Division, Corps of Engineers, Cincinnati, Ohio

James A. Neill, Chief of Engineering Division, Pittsburgh District, Corps of Engineers, Pittsburgh, Pa.

Dwight W. Keller, Civil Engineer, Planning and Reports Branch, Ohio River Division, Corps of Engineers, Cincinnati, Ohio

"Current Trends in Traffic and Equipment on the Ohio River and its Tributaries"

Charles F. Michiels, Chief of Operations Division, Ohio River Division, Corps of Engineers, Cincinnati, Ohio

William F. Lail, Assistant Chief of Operations Division, Ohio River Division, Corps of Engineers, Cincinnati, Ohio

Robert E. Mytinger, Chief of Plant and Equipment Branch, Ohio River Division, Corps of Engineers, Cincinnati, Ohio

"Flood Control Plan for the Ohio River Basin"

Bruce R. Gilcrest, Chief of Hydraulics Branch, Ohio River Division, Corps of Engineers, Cincinnati, Ohio

Emil P. Schuleen, Assistant Chief of Engineering Division, Pittsburgh District, Corps of Engineers, Pittsburgh, Pa.

Edgar W. Landenberger, Assistant Chief of Planning and Reports Branch, Ohio River Division, Corps of Engineers, Cincinnati, Ohio

"Local Flood-Protection Projects, Ohio River Basin"

Samuel M. Bailey, M. ASCE, Chief of Engineering Division, Louisville District, Corps of Engineers, Louisville, Kentucky

Harry Pockras, Chief of Engineering Division, Huntington District, Corps of Engineers, Huntington, West Virginia

Monday, October 15, 1956 - Afternoon Session

Presiding: Charles M. Wellons, Chairman, Executive Committee, Waterways and Harbors Division

"Design and Operating Characteristics of Roll-On Roll-Off Type Vessels"

Douglas C. MacMillan, President, George G. Sharp, Inc., New York City

"Shoreside Facilities for Roll-On Roll-Off and Other Special Purpose Ships"

Howard J. Marsden, Chief, Division of Port Development, Maritime Administration, U. S. Department of Commerce, New York, New York

"Master Plan for Redevelopment of Two Miles of Obsolete Waterfront"

Duncan S. Reid, Manager, Planning Division, Marine Terminals Department, Port of New York Authority, New York, New York

"Design and Utilization of Shipping Containers"

Donald A. Booth, M. ASCE, Assistant Chief Draftsman, Dravo Corp., Pittsburgh, Pa.

Tuesday, October 16, 1956 - Morning Session
(Jointly sponsored by Waterways and Harbors Division and Hydraulics Division)

Presiding: Charles M. Wellons, Chairman, Executive Committee, Waterways and Harbors Division

"The Littoral Drift Problem at Shoreline Harbors"

J. W. Johnson, Professor of Hydraulic Engineering, University of California, Berkeley, California

"Hurricane Wave Design Practices"

C. L. Bretschneider, A. M. ASCE, Research Engineer, Beach Erosion Board, U. S. Corps of Engineers, Washington, D. C.

Dr. B. W. Wilson, A.M. ASCE, Associate Professor, A&M College of Texas, Dept. of Oceanography, College Station, Texas

"Wave Forces on Cylindrical Piles"

R. L. Wiegel, Associate Research Engineer, Institute of Engineering Research, University of California, Berkeley, California

K. E. Beebe, Lt., USAR Ordnance Corps, Berkeley, California

Tuesday, October 16, 1956 - Luncheon
(Jointly sponsored by Waterways and Harbors Division, Structural Division and Hydraulics Division)

Address by Lt. Gen. Samuel D. Sturgis, Chief of Engineers, on the new construction underway and proposed for the Ohio River system.

Tuesday, October 16, 1956 - Afternoon Session

Presiding: Wallace M. Lansford, Member Executive Committee, Hydraulics Division

"An Approach to Hurricane Tide Prediction"

Robert L. Reid, Professor, Dept. of Oceanography, Texas A&M College, College Station, Texas

"The Effect of Hurricanes on Sea Level in Charleston Harbor"

Bernard Zettler

"The Use of Scale Models in Predicting Wind Tides"

J. W. Johnson, Professor of Hydraulic Engineering University of California, Berkeley, California

O. V. Sibul

JACKSON CONVENTION

Col. L. B. Feagin, Chairman of the Division Committee on Session Programs, has appointed Mr. Raymond W. Sauer of the Vicksburg District of the Corps of Engineers to the Committee. Mr. Sauer will be in charge of the development of the program for the Division for the Jackson, Mississippi, convention in February 1957.

DIVISION MEMBERSHIP

In November 1955, 290 ASCE members were registered in the Waterways and Harbors Division. By May of this year, the Division registration had increased to 400 and it is estimated that, at the present time, there are about 450 members registered in the Division.

AMERICAN ASSOCIATION OF PORT AUTHORITIES ANNUAL MEETING

The AAPA will hold its annual meeting in Philadelphia from September 18 to 21.

DIVISION COMMITTEE ON RESEARCH

The formation of the new Division Committee on Research has been completed, as follows:

Brig. Gen. H. D. Vogel (Chairman)
Chairman of the Board,
Tennessee Valley Authority
Knoxville, Tennessee

Mr. Howard J. Marsden
Chief, Division of Port Development
Maritime Administration
U. S. Department of Commerce
Washington 25, D. C.

Mr. Frank W. Edwards
Stanley Engineering Company
Chicago, Illinois

Colonel Andrew P. Rollins, Jr. (Secretary)
Director, U. S. Waterways Experiment Station
Vicksburg, Mississippi

Professor Joe W. Johnson
Professor of Hydraulic Engineering
University of California
Berkeley, California

The initial meeting of the Committee was held on Tuesday, June 5, 1956, in the office of the Chairman, and all members except Professor Johnson were present. Mr. Charles M. Wellons, Chairman, Executive Committee, Waterways and Harbors Division, and Mr. Don Reynolds, Assistant to the Secretary, ASCE, attended the latter part of the meeting and offered a number of valuable suggestions.

The Committee discussed the fields of research connected with waterways and harbors that would be of interest. Mr. Marsden gave a very interesting review of the mission of the Maritime Administration and its interest in research. He stated that there is a critical requirement for the development of design criteria for general cargo handling equipment at marine terminals. Further discussions covered the types of research presently underway and means available to the Committee to encourage such research as well as initiate new research. It was felt that one important function of the Committee would be to provide a means of making the results of such research more widely known and available.

NOTES FROM LOCAL SECTION REPRESENTATIVES

Professor Andre L. Jorissen, Head of the Department of Hydraulics and Hydraulic Engineering at Cornell University and Ithaca Section representative of the Committee on Cooperation with Local Sections, reports that he recently attended the Munich meeting of Technical Committee 30 of the International Organization for Standardization. This committee is studying the problem of standardization of methods and instruments for the measurement of fluid flow in pipe lines and open channels.

Mr. Walter F. Lawlor, Chief, Engineering Division, St. Louis District, Corps of Engineers and St. Louis Section representative of the Committee on Cooperation with Local Sections, reports that Col. Raymond M. Clock, formerly Executive Officer in the St. Louis District, has been reassigned to Fort Campbell, Kentucky.

Mr. Herbert C. Gee of Gee and Jenson, Inc., and Miami Section representative of the Committee on Cooperation with Local Sections reports that the committee appointed by the National Rivers and Harbors Congress to study the future of the Panama Canal made its final report at the last annual meeting of the Congress. The report has now been printed as a Congressional Document. The members of this committee are Henry H. Buckman, Adm. Paulus Powell, Perry Ford, Homer Angell, and Herbert C. Gee.

Kenneth P. Pell of the South Pacific Division, Corps of Engineers, has replaced Owen G. Stanley as Chairman of the Waterways and Harbors Division of the San Francisco Section.

PLANS FOR FUTURE NEWSLETTER ISSUES

The Division Executive Committee and I are attempting to make future issues of the Newsletter more interesting to Division members. To accomplish this, the Newsletter could include such material as reports of Waterways and Harbors Division meetings of local sections and news of the activities, promotions, transfers, etc., of Division members. Probably there is a great deal of other material which could be included.

The local section representatives of the Division have already been asked to send me any material which they think should be included in the Newsletter.

I would like to thank those who have furnished material for this issue. I would also like to ask all members of the Division to send me material for the Newsletter. I would appreciate any suggestions of Division members for improving the Newsletter.

Austin E. Brant, Jr.
Newsletter Editor

Tippetts-Abbett-McCarthy-Stratton
62 West 47th Street
New York 36, New York

[illegible]

[illegible]

Professor [illegible], Head of the Department of [illegible] Engineering at Cornell University [illegible] Local Sections, [illegible] Committee [illegible] methods and instruments [illegible]

[illegible] Chief, [illegible] Corps of Engineers [illegible]

[illegible]

[illegible]

[illegible]

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

VOLUME 81 (1955)

SEPTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)[c], 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)[c], 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)[c], 808(IR)[c].

OCTOBER: 809(ST), 810(HW)[c], 811(ST), 812(ST)[c], 813(ST)[c], 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)[c], 820(SA), 821(SA), 822(SA)[c], 823(HW), 824(HW).

NOVEMBER: 825(ST), 826(HY), 827(ST), 828(ST), 829(ST), 830(ST), 831(ST)[c], 832(CP), 833(CP), 834(CP), 835(CP)[c], 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)[c].

DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

VOLUME 82 (1956)

JANUARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1) 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877 (HW1)[c], 878(ST1)[c].

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1) 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904 (PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913 (WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941 (SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958 (SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969 (WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

JUNE: 990(PO3), 991(PO3), 992(PO3), 993(PO3), 994(PO3), 995(PO3), 996(PO3), 997(PO3), 998 (SA3), 999(SA3), 1000(SA3), 1001(SA3), 1002(SA3), 1003(SA3)[c], 1004(HY3), 1005(HY3), 1006 (HY3), 1007(HY3), 1008(HY3), 1009(HY3), 1010(HY3)[c], 1011(PO3)[c], 1012(SA3), 1013(SA3), 1014(SA3), 1015(HY3), 1016(SA3), 1017(PO3), 1018(PO3).

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026 (SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048 (SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068 (WW4)[c], 1069(WW4).

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c. Discussion of several papers, grouped by Divisions.